DISCLAIMER

Although the material in this Wisconsin Bridge Manual has been tested by the Bureau of Structures, no warranty, expressed or implied, is made by the Wisconsin Department of Transportation, as to the accuracy of the material in this manual, nor shall the fact of distribution constitute any such warranty, and responsibility is not assumed by Wisconsin Department of Transportation in connection therewith.

1.1 Introduction

The Bridge Manual is for the guidance of design engineers, technicians, and inspection personnel engaged in bridge design, plan preparation, and construction for the Wisconsin Department of Transportation. It is prepared to encourage uniform application of designs and standard details in plan preparation of bridges and other related structures.

This manual is a guide for the layout, design and preparation of highway structure plans. It does not replace, modify, or supersede any provisions of the Wisconsin Standard Specifications, plans or contracts.

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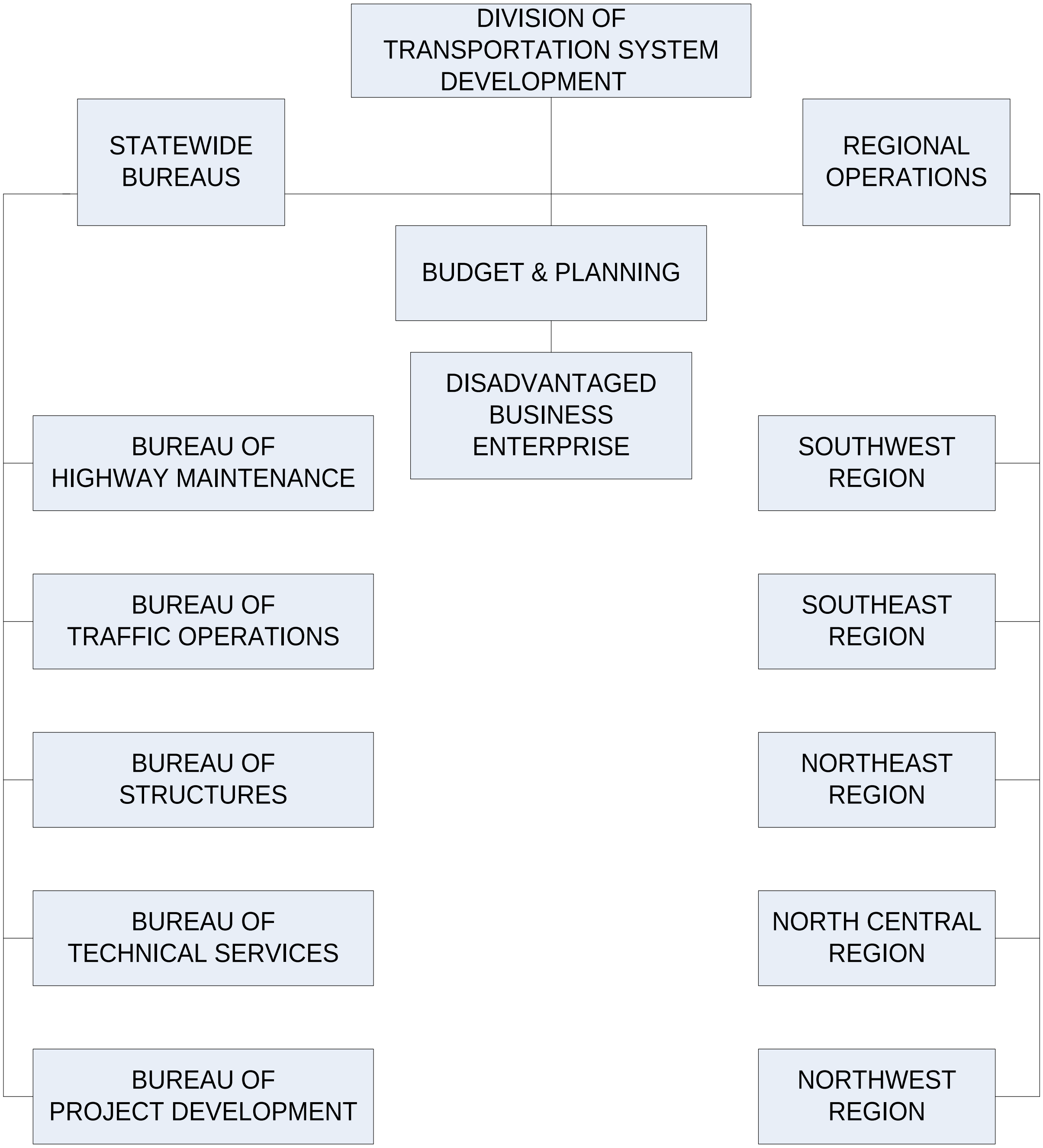


Figure 2.1-1
Division of Transportation System Development

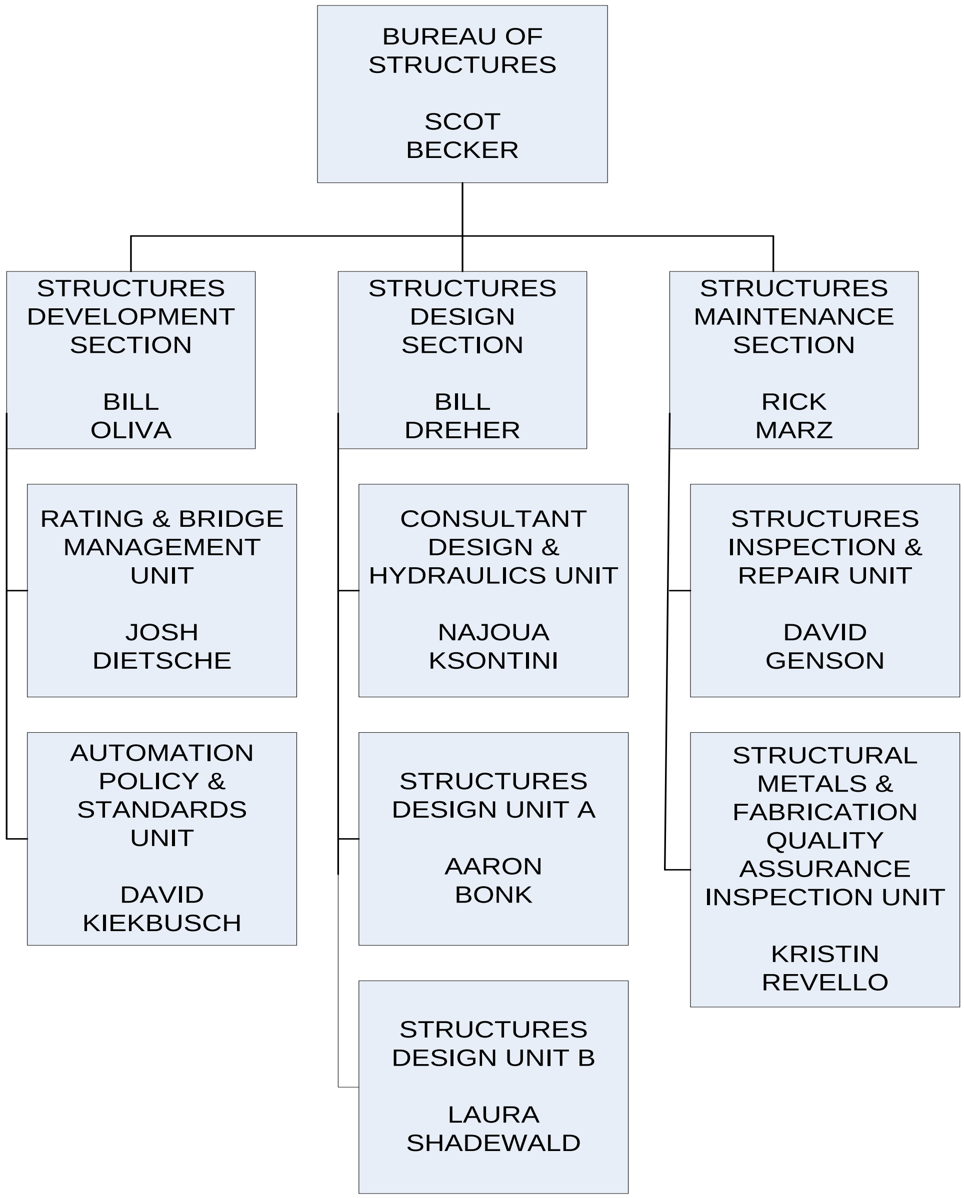
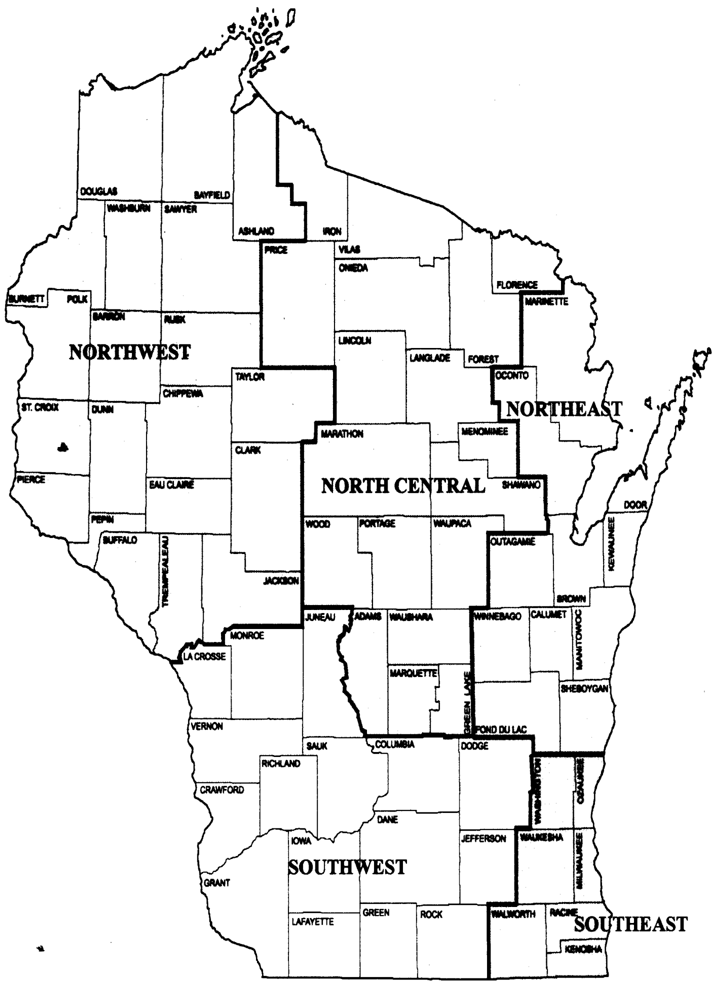


Figure 2.1-2
Bureau of Structures



NO.	COUNTY	REGION	NO.	COUNTY	REGION
1	ADAMS	NORTH CENTRAL	37	MARATHON	NORTH CENTRAL
2	ASHLAND	NORTHWEST	38	MARINETTE	NORTHEAST
3	BARRON	NORTHWEST	39	MARQUETTE	NORTH CENTRAL
4	BAYFIELD	NORTHWEST	40	MILWAUKEE	SOUTHEAST
5	BROWN	NORTHEAST	41	MONROE	SOUTHWEST
6	BUFFALO	NORTHWEST	42	OCONTO	NORTHEAST
7	BURNETT	NORTHWEST	43	ONEIDA	NORTH CENTRAL
8	CALUMET	NORTHEAST	44	OUTAGAMIE	NORTHEAST
9	CHIPPEWA	NORTHWEST	45	OZAUKEE	SOUTHEAST
10	CLARK	NORTHWEST	46	PEPIN	NORTHWEST
11	COLUMBIA	SOUTHWEST	47	PIERCE	NORTHWEST
12	CRAWFORD	SOUTHWEST	48	POLK	NORTHWEST
13	DANE	SOUTHWEST	49	PORTAGE	NORTH CENTRAL
14	DODGE	SOUTHWEST	50	PRICE	NORTH CENTRAL
15	DOOR	NORTHEAST	51	RACINE	SOUTHEAST
16	DOUGLAS	NORTHWEST	52	RICHLAND	SOUTHWEST
17	DUNN	NORTHWEST	53	ROCK	SOUTHWEST
18	EAU CLAIRE	NORTHWEST	54	RUSK	NORTHWEST
19	FLORENCE	NORTH CENTRAL	55	ST CROIX	NORTHWEST
20	FOND DU LAC	NORTHEAST	56	SAUK	SOUTHWEST
21	FOREST	NORTH CENTRAL	57	SAWYER	NORTHWEST
22	GRANT	SOUTHWEST	58	SHAWANO	NORTH CENTRAL
23	GREEN	SOUTHWEST	59	SHEBOYGAN	NORTHEAST
24	GREEN LAKE	NORTH CENTRAL	60	TAYLOR	NORTHWEST
25	IOWA	SOUTHWEST	61	TREMPEALEAU	NORTHWEST
26	IRON	NORTH CENTRAL	62	VERNON	SOUTHWEST
27	JACKSON	NORTHWEST	63	VILAS	NORTH CENTRAL
28	JEFFERSON	SOUTHWEST	64	WALWORTH	SOUTHEAST
29	JUNEAU	SOUTHWEST	65	WASHBURN	NORTHWEST
30	KENOSHA	SOUTHEAST	66	WASHINGTON	SOUTHEAST
31	KEWAUNEE	NORTHEAST	67	WAUKESHA	SOUTHEAST
32	LA CROSSE	SOUTHWEST	68	WAUPACA	NORTH CENTRAL
33	LAFAYETTE	SOUTHWEST	69	WAUSHARA	NORTH CENTRAL
34	LANGLADE	NORTH CENTRAL	70	WINNEBAGO	NORTHEAST
35	LINCOLN	NORTH CENTRAL	71	WOOD	NORTH CENTRAL
36	MANITOWOC	NORTHEAST	73	MENOMINEE	NORTH CENTRAL

Figure 2.1-3
Region Map

2.2 Incident Management

The procedures to be followed in Incident Management are:

2.2.1 Bridge Incidents

For Bridge Incidents such as vehicle hits on girders, column or railing that are not likely to cause a bridge failure, incident management will be handled by the Regional Office and after consultation of the Regional Duty Officer (RDO) the structures technical expert. Assistance will be provided by the Structures Design Section if rehabilitation plans are required. Refer to the contacts list below for names and telephone numbers.

WisDOT policy item:
The Bureau of Structures has an on call technical expert number that is to be activated with the consultation of the RDO for bridge incidents. The number is: (608) -206 -1280.

2.2.2 Major Bridge Failures

A bridge failure requires emergency action on the part of the DOT to protect the safety of other drivers and to prevent additional crashes, to establish appropriate traffic detours, to assess the damage, to determine its cause, to plan and implement its repair. Examples include a bridge that collapses as the result of flooding, being struck by a motor vehicle, or the weakening of its members. The Bureau of Structures follows all responsibilities and actions established in the Departments Emergency Operations Plan (ETO).

To organize and execute an effective and efficient response to a major bridge failure, DOT will follow the principles of National Incident Management System (NIMS) and incident command system (ICS). NIMS and ICS are being used nationwide as an organizational tool for command, control and coordination of responses to an array of incidents – large and small, natural and man-made, spontaneous or planned.

NIMS and ICS provide a common organizational structure and terminology within a unified command under the direction of an incident commander. A large-scale incident may require the establishment of a unified command to coordinate the activities of multiple jurisdictions.

When an incident takes place, law enforcement will generally be the first responders and will take steps to close the highway to traffic. County highway officials will erect barricades and begin detouring traffic.

The State Patrol or other law enforcement will contact the Regional Office and RDO who in turn should contact the Bureau of Structures technical expert. While the Regional Office will probably retain general oversight of the incident, a Bureau of Structures representative should be given shared responsibility.

The ICS structure recognizes the importance of public information and media relations during a high-profile incident. As part of the command staff, the public information officer (PIO) reports directly to the incident commander, who has the overall responsibility and authority for the response.

Generally, the PIO is responsible for coordinating the release of information to the media, handling reporters' inquiries and advising the incident commander about communication strategies. Public information in the event of a major bridge failure or damage will be directed at assuring the public that the DOT is taking appropriate action to protect other drivers, provide adequate routes, investigate the cause and make repairs in a timely manner.

The PIO may need assistance from the Bureau of Structures or region staff to provide: 1) details of the incident, 2) acknowledgement of deaths or injuries, 3) traffic detours, 4) plans for investigation, 5) cause of the incident, 6) plans to repair or rebuild the bridge, 7) maps locating the bridge, closed highways and alternate routes, 8) background information on the bridge's design, construction and past repairs, 9) recent bridge inspection reports, and 10) policies regarding bridge inspection.

The incident commander has the final authority over the release of all information to the media and the general public. DOT employees are authorized to make only the following statement when questioned in any capacity about emergency and recovery efforts. This policy and statement applies to all agency personnel at all offices:

> "Wisconsin DOT has activated an incident command system. All inquiries for more information are being handled by the information officer at our agency command center".

Only the information officer or incident commander is authorized to provide other information.

2.2.3 Bureau of Structures Actions in Incident Response

1. Document details from Regional Office RDO at time of contact.

2. Notify Bureau Director, Division Administrator, Secretary's Office & all Bureau coworkers via PCR (Public Communication Record).

3. In the event of a structure hit that compromises the ability to carry traffic, the DMV OSOW permitting unit should be notified.

4. Respond to the bridge site if requested.

    a. Determine In-house expertise for Project team.

    b. Determine if Consultant expertise is needed.

    c. Involve all available Bureau Sections in decision making.

    d. Have bridge plans available.

e. Establish a Bureau contact for communication from Response site.

f. Select at least one other structure person to go to the bridge site.

5. Observe all safety rules at bridge site.

6. Continue to communicate with all Bureau staff.

    a. Select at least one other structure person to go to Notify Bureau Contact Person to perform required communication.

7. Document actions taken and file for future reference. Communicate to all indicated in Item #2.

2.2.4 Public Communication Record

A "Public Communication Record": (PCR) is a form filled out by DOT employees to inform upper management and other potentially interested staff of a contact that may be of interest to the recipient. The contact is normally from the Media, Legislator, Local Official or the public concerning a topic that is or could be controversial now or in the future.

Within the Bureau of Structures (BOS), the Bureau Director, Section Managers, Supervisors and Lead workers should be included in all PCRs filled out by BOS staff, along with the established list of PCR contacts found in Outlook "Global Address List" under DOT DL PCR. A copy of the PCR form (DT 33) can be found on the DOTNET at:

http://dotnet/opa/opapolicies.htm

If you are contacted by the Media, Legislator or Local Official and are not sure if you need to fill out a PCR, contact your Supervisor for their opinion. A PCR is quick and easy to do so "if in doubt fill it out" is the best approach to use.

2.3 Responsibilities of Bureau of Structures

2.3.1 Structures Design Section

- Provide guidance to Regional Offices on the preparation of various types of Structure Survey Reports.
- Assist Regional Offices making design investigation studies by providing guidance on structure costs, depths, and practical structure types for the alternate sites under consideration.
- Prepare comparative cost estimates for alternate structure types. Prepare economic studies on rehabilitation versus replacement of existing structure. Make recommendations to Regional Office or Consultant or Government Agency.
- Review and approve Consultant preliminary and final plans, evaluate hydraulic adequacy and compliance to current Standards.
- Review and approve Consultant rehabilitation proposals.
- Collect and make information available to Regional Offices for hydrology studies and new hydraulic developments by other agencies.
- Provide procedures for scour analysis of structures.
- Make field observations of the proposed site, gather additional information for hydraulic reports, and evaluate the general conditions of the site. Coordinate hydraulic impacts with DNR.
- Assemble data and prepare drawings as required by Coast Guard for permit applications to construct bridges over navigable streams. Assemble data as necessary and receive certification from the Corps of Engineers and other agencies exercising environmental control over the proposed structure improvement.
- Prepare preliminary structures plans for bridges. This includes designing, detailing, drafting, estimating, and checking as may be necessary to obtain approvals from other governmental agencies.
- Determine size and length of box culverts. Design and plot culvert plans for checking by staff.
- Distribute preliminary structure plans to Regional Offices for approval and utility contacts.
- Prepare final contract plans for bridges, box culverts and other structures which include designing, detailing, drafting, estimating and ensuring compliance with preliminary study report and Standard Specifications.

- Prepare Special Provisions for construction of bridges, box culverts, and other structures covering special items not on the contract plans or in Standard Specifications.
- Review and approve permits relating to placement of utilities on structures.
- Evaluate bridges for rehabilitation, replacement or widening and recommend the course of action. Prepare contract plans for structure rehabilitation.
- Provide design and plans for bridge damage repair, contract change orders and steel repair.
- Provide technical assistance to Regional Offices or consultants with inquiries on final plans, specifications, materials, etc. in both the design and construction phases of the project.
- Upon request review construction falsework plans for structures.
- Design and prepare plans for sign bridges, sign supports, light poles, and other sign or lighting related to structures.
- Review fabrication drawings for monotube, highmast light towers, misc. light and sign support structures as submitted by Bureau of Highway Operations (Traffic Engineering Section) or Regional Office Traffic personnel.
- Make recommendations for standard bridge details, design procedures, and new computer programs to the Structures Development Section.
- Provide design costs for structures on an as needed basis to the Regional Offices for use in negotiating consultant contracts and budgeting in-house design time on structural projects.

2.3.2 Structures Development Section

- Create and maintain plan insert sheets that detail commonly used bridge components (i.e. Parapets, Railings, Bearings, Girders and Diaphragms).
- Review and approve overweight vehicle permit requests for State Trunk Network System bridges.
- Maintain the filing system and supervise the scanning of all highway structures data.
- Maintain the Highway Structures Information System (HSIS) for transportation structures.
- Develop and maintain Bridge Management Systems.

- Evaluate, implement, and develop new transportation structures Computer Programs and maintain all computer program documentation.

- Provide technical assistance and operational procedures for Bridge Engineering Workstations, CADDS and PC applications.

- Research, evaluate, and recommend the use of new materials, design theories, and structural types. Work closely with other transportation structure agencies and manufacturers in these areas gathering relevant facts and make recommendations for improving materials or product specifications.

- Develop and maintain text and tables for the Bridge Manual and post on extranet site.

- Develop and maintain Bridge Standard details by evaluating new design and/or by revising existing procedures, materials, and specifications. Prepare design tables, graphs, or curves to assist in structures design and detail plans preparation.

- Provide structure related technical assistance to Consultants, Contractors, Counties, DOT Central Offices, and Regional Offices with an emphasis on quality improvement of materials and/or procedures.

- Initiate work plans and provide specifications for Experimental Construction Projects. Provide follow up in-service inspection and performance evaluation reporting on new materials or methods.

- Maintain the Bridge Computer Programs as they relate to analysis and design procedures, materials, and specifications.

- Maintain office facilities for computer program documentation, manual texts and technical libraries for design, research records and new products information.

- Provide technical development, guidance, or review of material specifications such as AASHTO, ACI, ASTM, AWS, etc. in areas related to transportation structures.

2.3.3 Structures Maintenance Section

- Perform complex in-depth inspection of structures

- Write in-depth inspection reports

- Perform complex emergency repairs

- Perform routine inspections with or without special equipment

- Assist in bridge repair.

- Provide bridge inspection training courses

- Manage the bridge painting program

2.4 Bridge Standards and Insert Sheets

Bridge standards are drawings which show the standard practice for details used by WisDOT. These Standards have been developed over time by input from individuals involved in design, construction and maintenance. They are applicable to most structures and should be used unless exceptions are approved by the Section Managers.

The Insert Sheets represent the Standards and are intended to be used with minimum revision for insertion in the final set of plans for construction purposes.

1. FHWA Approval of Structure Standards Process

   The following points define the working relationship between FHWA and WISDOT concerning production and adoption of Bureau of Structures (BOS) Standard Detail Drawings. These points were agreed upon at a meeting on December 17, 2002 between BOS and FHWA.

- Submittals will be sent by electronic methods in PDF format to FHWA. (For special cases with a large amount of supporting information other methods may be used as agreed to by both parties on a case by case basis).

- Generally two weeks should be sufficient to render an approval or request for additional information. (In special cases requiring input from sources outside of the Wisconsin FHWA office additional time will be requested in writing with an expected due date for a decision agreed to by both WisDOT and FHWA).

- Appropriate supporting documentation ranging from written explanations to fully detailed engineering calculations will accompany submittals. The level of support should reflect the level of review expected.

- The Structure Standards reviewed by the FHWA will be done so with respect to Federal Law, Policy and safety issues. Differing opinions on other issues will not be cause for non-approval of standards.

2.5 Structure Numbers

An official number referred to as the Structure Number is assigned to every structure on the State Highway System for the purpose of having a definite designation. The Structure Number is hyphenated with the first letter being either a B, C, P, S, R, M or N. B is assigned to all structures over 20 ft. in length, including culvert configurations. In general, C is assigned to structures 20 ft. or less in length with an exception being that box culverts must have a cross sectional area greater than, or equal to 20 square feet to be assigned a C number. Do not include pipes unless they meet the definition of a bridge. A set of nested pipes may be given a Bridge Number if the distance between the outside walls of the end pipes exceeds 20 ft. and the clear distance between pipe openings is less than half the diameter of the smallest pipe. P designates structures for which there are no structural plans on file. S is for sign structures, R is for Retaining Walls, and N is for Noise Barriers. M is for miscellaneous structures where it is desirable to have a plan record. Bridges on state boundary lines also have a number designated by the adjacent state.

WisDOT Policy Item:
No new P numbers will be assigned as we should always request plans.

Regional Offices should assign numbers to structures before submitting information to the Bureau of Structures for the structural design process or the plan review process. Unit numbers are only assigned to long bridges or complex interchanges where it is desirable to have only one bridge number for the site.

For guidance on inspection and documentation of state-owned small bridge structures (C-XX-XXX), see the Structures Inspection Manual.

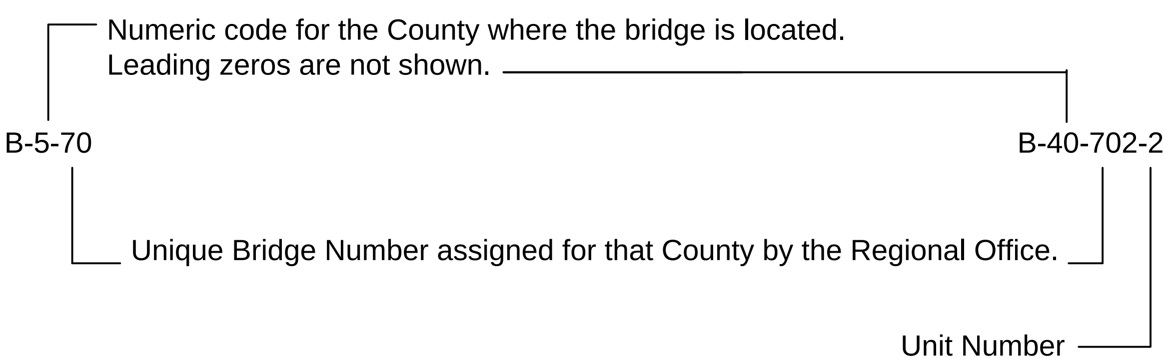


Figure 2.5-1
Bridge Number Detail

See 14.1.1.1 for criteria as to when a retaining wall gets assigned a R number and receives a name plate. A Structure Survey Report should be sent to the Structures Design Section, even if designed by the Regional Office.

See Section 6.3.3.7 for guidance on location of name plate on structures.

When a structure is rehabilitated, the name plate should be preserved, if possible, and reinstalled on the rehabilitated Structure. If a new name plate is required, it should show the year of original construction. The original structure number applies to all rehabilitation including widening, lengthening, superstructure replacement, etc.

Pedestrian only bridges get a B number if they are state maintained or cross a roadway. Otherwise use an M number for tracking purposes such as DNR bridges reviewed by DOT.

2.6 Bridge Files

Records and information useful in bridge planning and design are kept in appropriate places. Following is a brief summary of the various types of files, their contents and location. The data is arranged in alphabetical order for quick reference.

		Location	Agency
Bridge Cost Analysis		Structures Design	BOS
National Bridge Inventory Data			
	Information coded for the electronic computer file.	Structures Development	BOS
Catalogues		Structures Development	BOS
	Manufacturers' Product Files		
	Research Files and Technical Items		
Civil, Mechanical and Electrical Technical Reference Books			
Design Calculations			
	After project is completed, the design calculations are filed in a folder until they are digitized.	Bridge Files, Microfilm or in HSIS	BOS
Engineers' Estimates		-----------	BPD
FHWA Program Manual		-----------	BOS
Log of Test Borings		Geotechnical Section	BTS
	Records of all borings.		
	Borings for each bridge are kept in Bridge Folder or on microfilm.		
Manuals		Structures Development	BOS
Bridge Manual, Computer, Construction and Materials Manual, Design Manual, Maintenance Manual and Transportation Administrative Manual			
Maps		Structures Design	BOS
	Geological Maps, National Forests		
	Navigation Charts, Rivers-Harbors		
	State Park, Topographic, Historical		
Maps		Structures Development	BOS

	City-Village-Town (CVT) Maps showing location of bridges.		
Payment estimates to Contractors		-----------	BPD
ASTM Specifications		Structures Development	BOS
Plans		-----------	BOS
As built. All plans are digitized.		Structures Development	BOS
	Bridge Plans: Plans of structures designed but not yet advertised are in files.	-----------	BOS
Shop Plans of Active Steel Projects		Metals Fabrication and Inspection Unit	BOS
Records (Accounting)			
	Bridge Standards: Documentation for Standards and Bridge Manual	Structures Development	BOS
	Rainfall and Runoff Data	Structures Design	BOS
	Bids on Individual Items	____________________	BPD
Reports			
	Bridge Maintenance Reports	Structures Maintenance	BOS
	Federal Highway Experimental Project Reports	Structures Development	BOS
	Foundation Reports	Geotechnical Section	BTS
	Preliminary Reports: Contains Information necessary for Design of Structures.	-----------	Region
	Research Reports	Structures Development	BOS
Special Provisions of Active Projects		-----------	BOS
Specifications		Structures Development	BOS
	AASHTO, ACI, AWS, AREMA, AISC, CRSI, PTI, SSPC, etc.		
Survey Notes		-----------	Region
Text Books on Foundations, Structures and Bridge Design		Structures Development	BOS

Bureau Legend:

BOS - Bureau of Structures

BPD - Bureau of Project Development

BTS - Bureau of Technical Services

2.7 Contracts

Contracts are administered by construction personnel in the Regional Office where the project is located. The Bureau of Project Development coordinates the activities of the Regional Offices.

The contract contains the plans, specifications, supplemental specifications where applicable and special provisions where applicable. These parts of the contract are intended to be cooperative. In the event of a discrepancy, the Standard Specifications gives the priority part to be used.

2.8 Special Provisions

Special provisions are required for some projects to give special directions or requirements that are not otherwise satisfactorily detailed or prescribed in the standard specifications. Following are some of the principal functions of the special provisions:

1. Supplement the Standard Specifications by setting forth requirements which are not adequately covered, for the proposed project, by the Standard Specifications.

2. Alter the requirements of the Standard Specifications where such requirements are not appropriate for the proposed work.

3. Supplement the plans with verbal requirements where such requirements are too lengthy to be shown on the plans.

4. Call the bidder's attention to any unusual conditions, regulations or laws affecting the work.

5. For experimental use of a new material or system such as paint systems not covered in the Standard Specifications.

When preparing the special provisions for any project, the writer must visualize the project from the standpoint of the problems that may occur during construction.

Special provisions are generally written for a specific project or structure, however several "standard" bridge special provisions are available on-line at the Structures Design Information site:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/special-provisions.aspx

These special provisions may require modification to accurately reflect the requirements of individual projects or structures.

2.9 Terminology

AASHTO	American Association of State Highway and Transportation Officials.
ABUTMENT	Supports at the end of the bridge used to retain the approach embankment and carry the vertical and horizontal loads from the superstructure.
ACI	American Concrete Institute.
AISC	American Institute of Steel Construction.
Allowable Headwater	The maximum elevation to which water may be ponded upstream of a culvert or structure as specified by law or design.
Anchor Bolts	Bolts that are embedded in concrete which are used to attach an object to the concrete such as rail posts, bearings, etc.
ANSI	American National Standards Institute.
Apron	The paved area between wingwalls at the end of a culvert.
ASTM	American Society for Testing Materials.
ADT	Average Daily Traffic
Award	The decision to accept the proposal of the lowest responsible bidder for the specified work, subject to the execution and approval of a satisfactory contract bond and other conditions as may be specified or required by law.
AWS	American Welding Society.
Backfill	Fill materials placed between structural elements and existing embankment.
Backwater	An unnaturally high stage in a stream caused by obstruction of flow, as by a dam, a levee, or a bridge opening. Its measure is the excess of unnatural over natural stage. A back up of water due to a restriction.
Bar Chair	A device used to support horizontal reinforcing bars above the base of the form before the concrete is poured.
Bar Cutting Diagram	A diagram used in the detailing of bar steel reinforcement where the bar lengths vary as a straight line.
Base Course	The layer of specified material of designed thickness placed on a subbase or a subgrade to support a surface course.
Batter Pile	A pile that is purposely driven at an angle with vertical.
Bearings	Device to transfer girder reactions without overstressing the supports, insuring the bridge functions as intended. (See Fixed Bearings and Expansion Bearings).
Bearing Stiffener	A stiffener used at points of support on a steel beam to transmit the load from the top of the beam to the support point.
Bedrock	The solid rock underlying soils or other superficial formation.
Bench Mark	A relatively permanent object bearing a marked point whose elevation above or below an adopted datum is known.
Blocking Diagram	A diagram which shows the distance from a horizontal line to all significant points on a girder as it will be during erection.

Bridge	A structure having a span of more than 20 ft. from face to face of abutments, measured along the roadway centerline.
Bridge Approach	Includes the embankment materials and surface pavements that provide the transition between bridges and roadways.
Bushings	A lining used to reduce friction and/or insulate mating surfaces usually on steel hanger plate bearings.
Butt Splice	A splice where the ends of two adjoining pieces of metal in the same plane are fastened together by welding.
CADDS	Computer Aided Design and Drafting System.
Caisson	A watertight box of wood or steel sheeting; or a cylinder of steel and concrete, used for the purpose of making an excavation. Caissons may be either open (open to free air) or pneumatic (under compressed air).
Camber	A slight vertical curvature built into a structural member to allow for deflection and/or vertical grade.
Cathodic Protection	A method of protecting steel in concrete by impressing direct current via anodes thus making the bar steel cathodically protected.
Causeway	A raised road across wet or marshy ground or across water.
Change Order	A written order to the Contractor, signed by the Engineer, ordering a change in the work from that originally shown by the Plans and Specifications that has been found necessary. If the work is of a nature involving an adjustment or unit price, a Supplemental Agreement shall be executed. Change orders duly signed and executed by the Contractor constitute authorized modifications of the Contract.
City and Village Streets	City and Village streets are the public thoroughfares within the boundaries of incorporated municipalities. They are improved and maintained under the jurisdiction of the respective city and village authorities that constitute the local governing bodies. A few city and village streets are eligible for federal aid.
Cofferdam	A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within.
Composite Section	Two sections made of the same or different materials together to act as one integral section; such as a concrete slab on a steel or prestressed girder.
Compression Seals	A preformed, compartmented, elastomeric (neoprene) device, which is capable of constantly maintaining a compressive force against the joint interfaces in which it is inserted.
Concrete Overlay	1 1/2” to 2” of concrete placed on top of the deck, used to extend the life of the deck and provide a good riding surface.
Construction Limits	The Stations at which construction begins and ends.
Contract Time	The number of calendar days shown in the proposal which is allowed for completion of the work.

Contraction Joint	A joint in concrete that does not provide for expansion but allows for contraction or shrinkage by the opening up of a crack or joint.
Coordinates	Linear or angular dimensions designating the position of a point in relation to a given reference frame. In Wisconsin it refers to the State Plane Coordinate System.
County Trunk Highway System	The County Trunk Highway System, established in 1925, which forms the secondary system of highways within the State, constitutes the interconnecting highways of the State Trunk System, and is made up mainly of highways secondary in traffic importance. It consists generally of highways of local service and is improved and maintained by the 72 county boards, which constitute the local governing authorities. Many county trunks are eligible for federal aid.
Creep	Time dependent inelastic deformation under elastic loading of concrete or steel resulting solely from the presence of stress.
Cross Bracing	Bracing used between stringers and girders to hold them in place and stiffen the structure.
Culvert	A structure not classified as a bridge having a span of 20 ft. or less spanning a watercourse or other opening on a public highway.
Curb	A vertical or sloping member along the edge of a pavement or shoulder forming part of a gutter, strengthening or protecting the edge, and clearly defining the edge of vehicle operators. The surface of the curb facing the general direction of the pavement is called the "face".
Cut-Off-Wall	A wall built at the end of a culvert apron to prevent the undermining of the apron.
Dead Load	The weight of the materials used to build the structure including parapets, utilities and future wearing surface on deck.
Deadman	A concrete mass, buried in the earth behind a structure, that is used as an anchor for a rod or cable to resist horizontal forces that act on the structure.
Deck Structure	A structure that has its floor resting on top of all the main stress carrying members.
Deflection Joint	A joint placed in the parapets of bridges to prevent cracking of the parapet due to deflection of the superstructures.
Design Volume	A volume determined for use in design representing traffic expected to use the highway. Unless otherwise stated, it is an hourly volume.
DHV	Design hourly volume.
Diaphragm	A structural member used to tie adjoining girders together and stiffen them in a lateral direction as well as distribute loads.
Dolphins	A group of piles or sheet piling driven adjacent to a pier. Their purpose is to prevent extensive damage or possible collapse of a pier from a collision with a ship or barge.

Draped Strands	Strand pattern for prestressing strands, where strands are draped at the ends of the girder to decrease the prestressing stress where the applied moments are small.
Drift Pin	A metal pin, tapered at both ends, used to draw members of a steel structure together by being driven through the corresponding bolt holes.
Drip Groove	A groove formed into the underside of a projecting sill or coping to prevent water from following around the projection and reaching the face of the wall.
Dummy Joint	A groove in the surface of a concrete structure that resembles a joint but does not go all the way through. It provides a plane of weakness, and is used to ensure that any cracks that occur will be in a straight line.
Epoxy Coated Rebar	Bar steel reinforcement coated with a powdered epoxy resin to prevent corrosion of the bar steel.
Expansion Bearings	Bearings that allow longitudinal movement of the superstructure relative to the substructure and rotation of the superstructure relative to the substructure.
Expansion Device	A device placed at expansion points in bridge superstructures to carry the vertical bridge loads without preventing longitudinal movement.
Expansion Joint	An expansion device in concrete that allows expansion due to temperature changes, thereby preventing damage to the slabs.
Filler Plate	A steel plate or shim used to filling in space between compression members.
Fixed Bearings	Bearings that do not provide for any longitudinal movement of the superstructure relative to the substructure, but allows for rotation of the superstructure relative to the substructure.
Flat Slab	A reinforced concrete superstructure that has a uniform depth throughout.
Floor Beam	A transverse structural member that extends from truss to truss or from girder to girder across the bridge.
Fracture Critical Members	Steel tension members or steel tension components of members whose failure would probably cause a portion of or the entire bridge to collapse.
Fracture Mechanics	Study of crack growth in materials.
GVW	Gross vehicle weight which is the total weight of basic truck, body and related payload.
Geotextiles	Sheets of woven or nonwoven synthetic polymers or nylon used for drainage and soil stabilization.
Girder	Main longitudinal load carrying member in a structure.
Grade Separation	A crossing of two highways, or a highway and a railroad, at different levels.
Grid Floors	Prefabricated steel grids set on girders and/or stringers provide the roadway surface, generally on moveable highway structures.

Hammerhead Pier	A pier which has only one column with a cantilever cap and is somewhat similar to the shape of a hammer.
Hanger Plate	A steel plate which connects the pins at hinge points thus transmitting the load through the hinge.
Haunch	An increase in depth of a structural member usually at points of intermediate support.
Haunched Slab	A reinforced concrete superstructure that is haunched (has an increased depth) at the intermediate supports.
Hinge	A device used to hold the ends of two adjoining girders together, but allowing for longitudinal movement of the superstructure.
Hinged Bearing	At hinge location along a girder, where forces from supported member are transferred to supporting member by a bearing (See Std. 24.8).
Holddown Device	A device used on bridge abutments to prevent girders from lifting off their bearings as a result of the passage of liveload over the bridge.
Hybrid Girder	A steel plate girder with the web steel having a lower yield strength than the steel in one or both flanges.
Inlet Control	The case where the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including barrel shape, cross sectional area, and inlet edge.
Intermediate Stiffener	A vertical transverse steel member used to stiffen the webs of plate girders between points of supports.
Jetting	Forcing water into holes in an embankment to settle or compact the earth.
Laminated Elastomeric Bearing	A bearing device constructed from elastomer layers restraining at their interfaces by integrally bonded steel or fabric reinforcement. Its purpose is to transmit loads and accommodate movements between a bridge and its supporting structure.
Lateral Bracing	Bracing placed in a horizontal plane between steel girders near the bottom and/or top flanges.
Leads	The vertical members of a pile driver that steady the hammer and pile during the driving.
Liquid Penetrant Inspection	Nondestructive testing method that reveals surface discontinuities by the bleedout of a penetrating medium against a contrasting colored background.
Live Load	For highway structures AASHTO truck or lane loadings. The weight of moving loads.
LRFD	Load Resistance Factor Design.
Longitudinal Stiffener	A longitudinal steel plate (parallel to girder flanges) used to stiffen the webs of welded plate girders.
Low Relaxation Strands	Prestressing tendons which are manufactured by subjecting the strands to heat treatment and tensioning causing a permanent elongation. This increases the strand yield strength and reduces strand relaxation under constant tensile stress.

Low Slump Concrete	Grade "E" concrete, used for concrete masonry overlays and repairs on decks.
Mag Particle Inspection	Nondestructive testing method that is used primarily to discover surface discontinuities in ferro magnetic materials by applying dry magnetic particles to a weld area or surface area that has been suitably magnetized.
Modular Exp. Joints	Multiple, watertight units placed on structures requiring expansion movements greater than 4”.
Mud Sill	A timber platform laid on earth as a support for vertical members or bridge falsework.
NCHRP	National Cooperative Highway Research Program.
Negative Moment	The moment causing tension in the top fibers and compression in the bottom fibers of a structural member.
Negative Reinforcement	Reinforcement placed in concrete to resist negative bending moments.
Non-Redundant Structure	Type of structure with single load path, where a single fracture in a member can lead to the collapse of the structure.
Oil Well Pipe Pile	High quality pipe used in oil industry drilling operations that may be used as an alternate to HP piling.
Outlet Control	The case where the discharge capacity of a culvert is controlled by the elevation of the tailwater in the outlet channel and the slope, roughness, and length of the culvert barrel, in addition to the cross sectional area and inlet geometrics.
P S & E	Literally plans, specifications, and estimates. Usually it refers to the time when the plans, specifications, and estimates on a project have been completed and referred to FHWA for approval. When the P S & E have been approved, the project goes from the preliminary engineering phase to the construction phase.
Parapet	A masonry barrier designed and placed to protect traffic from falling over the edge of a bridge, or in some cases, from crossing lanes of traffic traveling in opposite directions.
Pier	Intermediate substructure unit of a bridge.
Pile	A long, slender piece of wood, concrete, or metal to be driven or jetted into the earth or river bed to serve as a support or protection.
Pile Bent	A pier where the piles are extended to the pier cap to support the structure.
Pile Cap	A slab, usually of reinforced concrete, covering the tops of a group of piles for the purpose of tying them together and transmitting to them as a group the load of the structure which they are to carry.
Pile Foot	The lower extremity of a pile.
Pile Head	The top of a pile.
Pile Points	Metal tip fastened to the lower end of pile to protect it when the driving is hard.
Pin Plate	A steel plate attached to the web plate of girders at hinge points to strengthen the web plate of girders at the hinge locations.

Positive Moment	The moment causing compression in the top fibers and tension in the bottom fibers in a structural member.
Post-Tensioned	Method of prestressing in which the tendon is tensioned after the concrete has cured.
Prestress Camber	The deflection in prestressed girders (usually upward) due to the application of the prestressing force.
Prestressed Concrete	Concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree. In reinforced-concrete members the prestress is commonly introduced by tensioning the steel reinforcement.
Pretensioned	Any method of prestressing in which the strands are tensioned before the concrete is placed.
Radiographic Inspection	Nondestructive testing method where gamma rays or X rays pass thru the object and cast an image of the internal structure onto a sheet of film as the result of density changes.
Redundant Structure	Type of structure with multi-load paths where a single fracture in a member cannot lead to the collapse of the structure.
Reflection Crack	A crack appearing in a resurface or overlay caused by movement at joints or cracks in underlying base or surface.
Residual Camber	Camber due to the prestressing force minus the deadload deflection of the girder.
RIPRAP	A facing of stone used to prevent erosion. It is usually dumped into place, but is occasionally placed by hand.
Rolled Girder Structure	A structure which has a rolled steel beam as the main stress carrying member.
Roughometer	A wheeled instrument used for testing riding qualities or road surfaces.
S.S.P.C.	Steel Structures Painting Council.
Semi-Retaining Abutment	An abutment used for retaining part of the back-fill of the roadway as well as supporting the end of the bridge.
Semi-Through Structure	A structure that has no overhead bracing, but the main stress carrying members project above the floor level.
Shear Connector	A connector used to join cast-in-place concrete to a steel section and to resist the shear at the connection.
Sheet Pile	A pile made of flat or arched cross section to be driven into the ground and meshed or interlocked with like members to form a wall, or bulkhead.
Shoulders	The portions of the roadway between the traveled way and the inside edges of slopes of ditches or fills, exclusive of auxiliary lanes, curbs, and gutters.
Shrinkage	Contraction of concrete due to drying and chemical changes, dependent on time.
Sill Abutment	A shallow concrete masonry abutment generally about 5 feet deep.
Simple Spans	Spans with the main stress carrying members non-continuous, or broken, at the intermediate supports.

Skew or Skew Angle	The acute angle formed by the intersection of a line normal to the centerline of the roadway with a line parallel to the face of the abutments or piers, or in the case of culverts with the centerline of the culverts. Left hand forward skew indicates that, look up station, the left side of the structure is further up station than the right hand side. Right hand skew indicates that the right side of structure is further up station than the left side.
Slope	The degree of inclination to the horizontal. Usually expressed as a ratio, such as 1:25 or 1 of 25, indicating 1 unit rise in 25 units of horizontal distance.
Slope Paving	Paving placed on the slope in front of abutment under a bridge to prevent soil erosion and sliding.
Spandrel	The area between the roadway and the arch in the side view of an arch bridge.
Special Provisions	Special directions and requirements that are prepared for the project under consideration and made a part of the contract.
Specifications	The body of directions, provisions, and requirements contained herein, together with written agreements and all documents of any description, made or to be made, pertaining to the method or manner of performing the work, the quantities, and the quality of materials to be furnished under the contract.
Spread Footing	A footing that is supported directly by soil or rock.
Spur Dike	A wall or mound built or extended out from the upstream side of an abutment used to train the stream flow to prevent erosion of stream bank. May also be used where there is no bridge, but the stream flows along the side of highway embankment.
Stainless Steel Teflon Bearings	Incorporates stainless steel and Teflon with steel to provide the necessary expansion movement.
State Plane Coordinates	The plane-rectangular coordinate system established by the United States Coast and Geodetic Survey. The plane coordinate system in Wisconsin is based on the Lambert conformal conic projection. Plane coordinates are used to locate geographic position.
State Trunk Highway Network	The system of highways heretofore selected and laid out by the Legislature and special legislative committees and by the Commission, and as revised, altered and changed by the Commission, including temporary routes designated by the Commission, the portions of the Interstate Highway System within the state, and routes adopted by the American Association of State Highway Officials as part of the U.S. Numbered Route System.
Stirrup	Vertical U-shaped or rectangular shaped bars placed in concrete beams to resist the shearing stresses in the beam.
Strip Seal Joint	Molded neoprene glands inserted and mechanically locked between armored interfaces of extruded steel sections.
Substructure	All of that part of the structure below the bridge seats or below the skewbacks of arches, or below the tops of the caps of piling or

	framed trestles, except that the wingwalls and parapets of abutments shall be considered as part of the substructure.
Superstructure	That part of the structure above the bridge seats, or above the skewbacks of arches, or above the tops of the caps of piling or framed trestles, including the flooring, but excluding wing walls and parapets of abutments (See substructures).
Supplemental Specifications	Specifications adopted subsequent to the publication of these specifications. They generally involve new construction items or substantial changes in the approved specifications. Supplemental specifications prevail over those published whenever in conflict therewith.
Surcharge	Any load that causes thrust on a retaining wall, other than backfill to the level of the top of the wall.
TRB	Transportation Research Board.
Temporary Holddown Device	A device used at the ends of steel bridges where the slab pour terminates to prevent the girders from lifting off the abutment bridge seats during the pouring of the concrete deck.
Tendon	A name for prestressed reinforcing element whether wires, bars, or strands.
Through Structure	A structure that has its floor connected to the lower portion of the main stress-carrying members, so that the bracing goes over the traffic. A structure whose main supporting members project above the deck or surface.
Tining	Used on finished concrete deck or slab surfaces to provide friction and reduce hydroplaning.
Town Road System	The town road system, or tertiary system of highways within the state, has been improved or maintained under the jurisdiction of the town boards, which are the local governing bodies. Some of the town roads are eligible for federal aid.
Transfer Stresses	In pretensioned prestressed concrete members the stresses that take place at the release of prestress from the bulkheads.
Ultrasonic Inspection	A non-destructive inspection process where by an ultra-high frequency sound wave induced into a material is picked up in reflection from any interface or boundary.
Unbonded Strands	Strands so coated as to prevent their forming a bond with surrounding concrete. Used to reduce stress at the ends of a member.
Underpinning	The adding of new permanent support to existing foundations, to provide either additional capacity or additional depth.
Uplift	A force tending to raise a structure or part of a structure and usually caused by wind and/or eccentric loads, or the passage of live-load over the structure.
Waterproofing Members	Impervious asphaltic sheets overlaid with bituminous concrete to protect decks from the infiltration of chlorides and resulting deterioration.

Wearing Surface	The top layer of a pavement designed to provide a surface resistant to traffic abrasion.
Weep Hole	A drain hole through a wall to prevent the building up of hydraulic pressure behind the wall.
Weir	A dam across a stream for diverting or measuring the flow.
Weld Inspection	Covers the process, written procedure, and welding in process. Post weld heat maintenance if required, post weld visual inspection and non-destructive testing as specified in contract and Standard Specifications.
Welded Wire Fabric	A two-way reinforcement system, fabricated from cold-drawn steel wire, having parallel longitudinal wires welded at regular intervals to parallel transverse wires and conforming to "Specifications for Welded Steel Wire Fabric for Concrete Reinforcement", AASHTO.
Well-Graded	An aggregate possessing proportionate distribution of successive particle sizes.
Wingwall	A wall attached to the abutments of bridges or box culverts retaining the backfill of the roadway. The sloping retaining walls on each side of the center part of a bridge abutment.

Table 2.9-1
Terminology

2.10 WisDOT Bridge History

Prior to the early 1950's, structure types on Wisconsin State Highways were predominantly reinforced concrete slabs and steel girders or trusses with reinforced concrete decks. Also, timber structures were used at a number of county and town road sites. In 1952, the first prestressed concrete voided slab sections were cast and erected incorporating transverse post-tensioning. In 1956, the first prestressed concrete "I" girders were designed and precast. After field setting, these prestressed girders were post-tensioned and completed with an integral cast-in-place reinforced concrete deck. During the mid-1950's and early 1960's, prestressed concrete "I" and steel girder structures were made continuous and incorporated composite designs for carrying live loads.

In 1971, the first cable-stayed bridge in the United States, a three span pedestrian structure, was constructed in Menomonee Falls.

2.10.1 Unique Structures

Structure Type	Bridge Number	Year Constructed	(feet) Span Configuration
Steel Rigid Frames	B-40-48-Milwaukee	1959	45.3, 168.5, 46.3
Steel Rigid Frames	B-56-47/48*-Mirror Lake	1961	50.6, 22-.0, 49.4
Overhead Timber Truss	B-22-50*-Cassville	1962	48.0
Arch Truss	B-16-5-Superior	1961	270.0, 600.1, 270.0
Tied Arches	B-9-87*-Cornell	1971	485.0
Tied Arches	B-12-27*-Prairie du Chien	1974	462.0
Tied Arches	B-40-400-Milwaukee	1974	270.0, 600, 270.0
Tied Arches	B-5-158*-Green Bay	1980	450.1
Tied Arches	B-22-60-Dubuque, IA	1982	670.0
Tied Arches	B-16-38*-Superior	1984	500.0
Prestressed "I" Girders with Cantilever	B-40-524*-Milwaukee	1985	112.0, 69.0, 107.8, 383.5 Spans with 25' Cantilevers
Prestressed Strutted Arches	B-40-603-Milwaukee	1992	8-158.0 Strutted Arch Spans
Tied Arches	B-32-202* - LaCrosse	2004	475'

Table 2.10-1
Unique Structures

* Designed in the Wisconsin Department of Transportation Bureau of Structures.

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3.1 Specifications and Standards

All bridges in the State of Wisconsin carrying highway traffic are to be designed to the *American Association of State Highway and Transportation Officials (AASHTO) LRFD Design Specifications,* the *American Society for Testing and Materials (ASTM)*, the *American Welding Society (AWS)* and Wisconsin Department of Transportation Standards. The material in this *Bridge Manual* is supplemental to these specifications and takes precedence over them.

All highway bridges are to be constructed according to State of Wisconsin, Department of Transportation, *Division of Transportation Systems Development Standard Specifications for Highway and Structure Construction* and applicable supplemental specifications and special provisions as necessary for the individual project.

All railroad bridges are to be designed to the specifications of the *American Railway Engineering Maintenance-of-Way Association (AREMA) Manual for Railway Engineering* and the specifications of the railroad involved.

3.2 Geometrics and Loading

The structure location is determined by the alignment of the highway or railroad being carried by the bridge and the alignment of the feature being crossed. If the bridge is on a horizontal curve, refer to Figure 3.2-1 to determine the method used for bridge layout. The method of transition from tangent to curve can be found in *AASHTO - A Policy on Geometric Design of Highways and Streets*. Layout structures on the skew when the skew angle exceeds 2 degrees; otherwise detail structures showing a zero skew when possible.

For highway structures, the minimum desirable longitudinal vertical gradient is 0.5 percent. There have been ponding problems on bridges with smaller gradients. This requirement is applied to the bridge in its final condition, without consideration of short term camber effects. Vertical curves with the high point located on the bridge are acceptable provided that sufficient grade each side of the high point is provided to facilitate drainage. Keeping the apex of the curve off of a pier, especially for slab bridges, can be beneficial to reduce ponding at those locations.

The clearances required on highway crossings are given in the *Facilities Development Manual* (FDM). The recommended clearance for railroad crossings is shown on Chapter 38 Standard for Highway Over Railroad Design Requirements. Proposed railroad clearances are subject to review by the railroad involved.

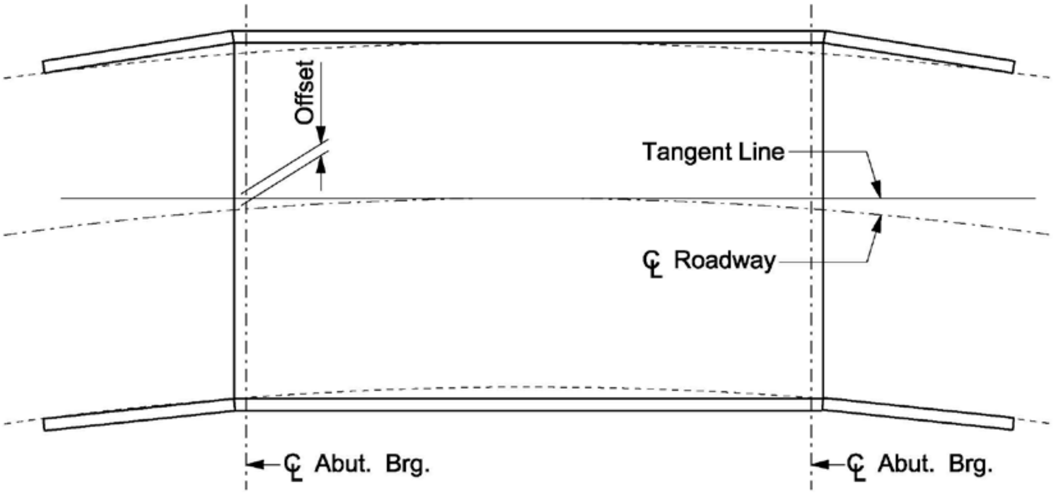
Highway bridge design live loadings follow the AASHTO LRFD Design specifications using HL93. Chapter 17 provides more detail on applying this load for design. WisDOT requires a specific vehicle design check using the Wis-SPV (Standard Permit Vehicle) which can be found in Chapter 45.

Railroad loadings are specified in the *AREMA Manual for Railway Engineering*.

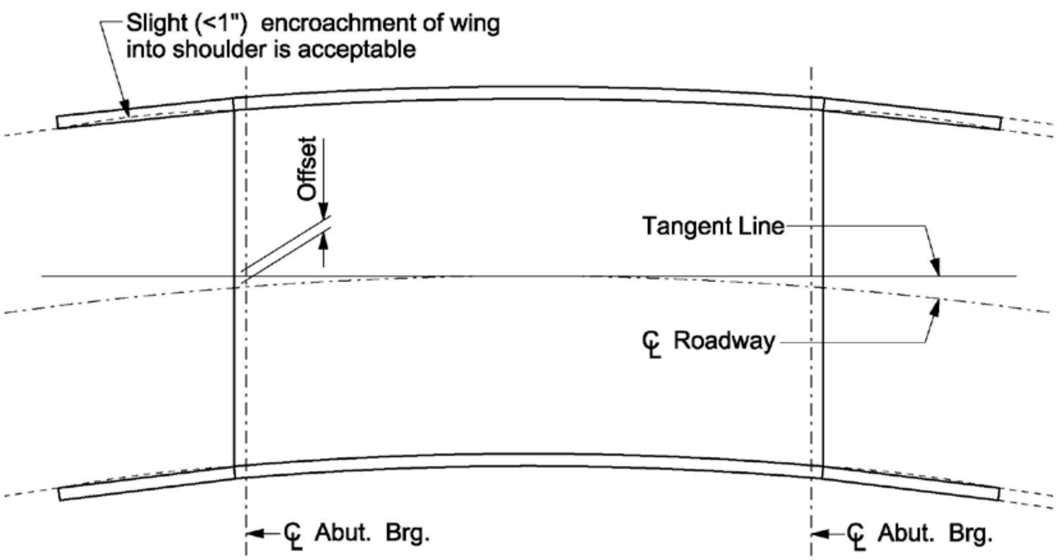
All new bridges constructed in the State of Wisconsin are designed for the clearances shown in FDM Procedure 11-35-1, Attachment 1.8. FDM Procedure 11-35-1, Attachment 1.9 covers the cases described in that section as well as bridge widenings. Wires and cables over highways are designed for clearances of 18’-0” to 22’-0”. Vertical clearance is needed for the entire roadway width (critical point to include traveled way, auxiliary lanes, turn lanes and shoulders).

Sidewalks on bridges shall be designed a minimum of 6 feet wide. Refer to the FDM for more details.

The length of bridge approaches should be determined using appropriate design standards. Refer to FDM 3.5.6 for discussion of touchdown points on local program bridge projects.



Case 1
For offsets 0" to 6"
Keep bridge straight. Widen bridge to provide full lane and shoulder width over entire length of bridge (round up to nearest 1"). Align straight wings so inside of wing tip is at edge of shoulder.



Case 2
For offsets over 6"
Curve entire bridge. Do not widen. Align straight wings so inside of wing tip is at edge of shoulder.

Figure 3.2-1
Bridge Layout on Horizontal Curves

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4.1 Introduction

Transportation structures, such as bridges and retaining walls, have a strong influence on the appearance of transportation projects, as well as the overall appearance of the general vicinity of the project. In locations where there is an opportunity to appreciate such structures, it is often desirable to add aesthetic enhancements to fit the project site.

Desirable bridge aesthetics do not necessarily need to cost much, if any, additional money. Aesthetic enhancements can be made in a number of ways. Primary features such as structure type and shape have the most influence on appearance, with color and texture playing secondary roles. Formliners, especially when used in conjunction with a multi-colored stain, are more expensive than one or two single color stains on smooth concrete, and have on a number of occasions not fit the context of the project. It is the responsibility of the design team to identify aesthetic treatments that are consistent with the environment and goals of the project, are maintainable over the life of the structures, and are cost effective. See 4.5 for current policy regarding structure aesthetics.

While initial cost for aesthetic enhancements is a concern, it has become apparent that maintenance costs can be considerably more than initial costs. Stain, which acts more like paint, must be periodically redone. Such reapplication oftentimes requires lane closures which are both an undesirable inconvenience to users and come with a significant cost associated with maintenance-of-traffic.

4.2 General Aesthetic Guidelines

Primary features – in relative order of importance:

- Superstructure type and shape, with parapets/railings/fencing being fairly prominent, as well. See Chapter 30 – Railings for further guidance.
- Abutment type and shape, with the wings being most prominent.
- Pier type and shape, with the end elevation being the most notable, especially for a bridge over a highway.
- Grade and/or skews.

Secondary features – in relative order of importance:

- Color
- Pattern and texture
- Ornamentation

Consider the following key points, in relative order of importance, when designing structures:

1. Simplicity
2. Good proportions with an emphasis on thinner members, or members that appear thinner
3. Clear demonstration of how the structure works with recognizable flow of forces
4. Fitting its context/surroundings
5. Good proportions in 3 dimensions
6. Choice of materials
7. Coloring – neutral colors, preferably no more than two. (Chapter 9 – Materials lists federal color numbers used most commonly for girders)
8. Pattern and texture
9. Lighting

Consider the bridge shape, relative to the form and function at the location. Use a structural shape that blends with its surroundings. The aesthetic impact is the effect made on the viewer by every aspect of a bridge in its totality and in its individual parts. The designer makes an aesthetic decision as well as a structural decision when sizing a girder or locating a pier.

The structure lines should flow smoothly with as few interruptions as possible. Do not clutter up the structure with distractive elements. If light standards are required, place them in line with the piers and abutments, so the vertical lines blend. Light spacing, however, needs to be coordinated with the Regional electrical engineer. Steel girder bearing stiffeners should be the only vertical stiffeners on the outside face of the exterior girders, although longitudinal stiffeners on the outside face can have an appealing look.

Refer to the WisDOT Traffic Guidelines Manual 2-1-60 for guidance on community sensitive design signing.

4.3 Primary Features

Superstructure Type and Shape

At highway speeds, highway structures are viewed from 300-500 feet away. The general shape of the bridge, with an emphasis on thinness, produces the most appealing structure. Given that there are realistic physical limitations on thinness (without resorting to anchored end spans or other costly measures), the designer has other options available to achieve the appearance of thinness such as:

- Larger overhangs to create better shadow lines.
- Horizontal recess on the backside of the parapet, which could be stained or left as plain concrete. Any parapet that is non-standard (either side) is considered CSS.
- Eliminate or minimize pedestals along the parapet. Such pedestals tend to break up the horizontal flow and make the superstructure appear top heavy. Pedestals, if desired, are better left on the wings to delineate the beginning or end of the bridge or to frame the bridge when viewed from below. If used on the superstructure, keep the pedestal size smaller and space apart far enough to avoid a top heavy appearance. See Chapter 30 – Railings for further guidance.
- Minimize vertical or patterned elements on the backside of the parapet as such elements tend to break up the horizontal flow. Rock formliner has become an overused aesthetic enhancement for the backside of parapets, as its use oftentimes does not fit the surroundings. Any parapet that is non-standard (either side) is considered CSS. See Chapter 30 – Railings for further guidance.
- Structure type should be based on economics, not aesthetics. Additional costs associated with a preferred structure type are considered CSS. Add-ons, such as false arches, etc. are considered CSS.

Abutment Type and Shape

Wing walls are the most visible portion of the abutment. Unless pedestrians are beneath a bridge, formliners or other aesthetic enhancements are not very visible and should be left off of the abutment front face, as these *treatments provide no additional aesthetic value.*

Pier Type and Shape

Pier shapes should be kept relatively simple and uncluttered. For highway grade separations, the end elevation of the pier is the view most often seen by the traveling public. For slower speed roads or where pedestrians travel beneath a bridge, the front pier elevation is also seen. For taller piers, such as those used for multi-level interchanges or water crossings, the entire 3D-view of the pier is readily seen and the pier shape is very important. For such piers, a clean, smooth flowing slender shape that clearly demonstrates the flow of forces from the superstructure to the ground is essential. External and internal (reentrant) corners on the pier/column shaft should be kept to a reasonable number. (Approximately 8 external, 4 internal maximum).

Grade and/or Skew

While grade and skew cannot be controlled by the bridge design engineer, these geometric features do affect bridge appearance. For example, a steep grade or pronounced vertical curve makes the use of a block type rustication an awkward choice. Horizontal blocks are typically associated with buildings and block buildings tend to have level roof lines. Cut stone form liners used on steep grades or pronounced vertical curves require excessive cutting of forms, which drives up price. Consideration of abutment height warrants more consideration when bridges are on steep grades, with a more exposed abutment face on the high end of the bridge producing a more balanced look.

Large skews tend to make piers longer as well as making the front elevation of the pier more visible to properties adjacent to the bridge. With larger skews, having more than one multi-columned pier can create a ‘forest’ of pier columns if the columns are too numerous. Try to maximize column spacing or use multiple hammerhead piers to help alleviate this effect. Abutment wings tend to be longer on the acute corners of bridges. Whatever aesthetic treatment is used needs to be appropriate for both the longer and shorter wings.

The design engineer should keep in mind that a bridge is never entirely seen at a 90-degree angle as depicted in a side elevation view. As the person viewing the bridge moves closer to the bridge the pier directly in front of them will be seen nearly as an end elevation of the pier, while adjacent piers will start to be viewed more as a pier side elevation. The ‘forest’ of columns starts to take effect, again, especially for wider bridges.

4.4 Secondary Features

Color

Color can have a strong visual effect, either positive or negative. Using earth toned colors versus vivid colors is preferred. More neutral colors tend to blend in more with the surroundings. Also, over time earth tones will weather less and not appear as dingy or faded. A bright yellow, for example, will begin to appear dull and dirty soon after application. Avoid red as this color is not UV tolerant and will fade. Concrete stain behaves more like paint and is susceptible to fading and peeling, requiring re-application to avoid an unsightly structure. Stained concrete in need of maintenance looks worse than concrete that was originally left unstained.

Using a maximum of two colors will lend itself to the desired outcome of a clean appearance. On larger structures it may be desirable to use two colors for everything other than the girders, which may be a third color. Remember that plain concrete is a color, too. It should be utilized as much as possible (especially on smaller surfaces) to reduce initial cost and, especially, future maintenance costs.

Utilizing a ribbed, or broken ribbed pattern on a large expanse of plain concrete can give the appearance of color as the patterned section will appear darker than the adjacent plain concrete. This is a good way to add ‘color’ without the future maintenance costs associated with actual stain reapplication.

As much as possible, Federal color numbers should be used for color selection. A few colors are given in Chapter 9 – Materials, but others may be used. STSP’s should be used as is for staining and multi-colored staining. Specific colors, areas to be applied, etc. should be referenced on the plan sheets.

Pattern and Texture

See 4.5 for current policy regarding structure aesthetics, including patterns and texture.

Large expanses of flat concrete, even if colored, are usually not desirable.

Most bridges are seen from below by people traveling at higher rates of speed. Detail smaller than 4-inches is difficult to discern. The general shape, and perhaps color, will have a greater visual effect than the pattern and/or texture. Sometimes texture is used to represent a building material that wasn’t used for the construction of the structure, as would be the case of rock form liner. While a rock appearance might be appropriate for a smaller bridge over a stream in a small town, it seldom fits the context of a grade separation over a highway or busy urban interchange. Modern bridges should, for the most part, look like they are built out of modern materials appropriate to the current time. Texture consisting of random or ordered geometric forms is generally more preferred over simulating other materials.

On MSE retaining walls it is desirable to keep logos or depictions within a given panel. Matching lines across panels, especially horizontal lines susceptible to differential panel settlement, is difficult. Rock texturing is unconvincing as real stone due to panel joints. A random geometric pattern is a good way to give relief to a wall.

Repetition in pattern rather than an assembly of various patterns or details is more cost effective. For effects that are meant to appear random (e.g. rock), care must be taken in order for the pattern repetition to not appear noticeable.

At all locations on a structure (abutment wings and piers, MSE walls, etc.), form details should be terminated 1'-0" below low water or ground elevations where they will not be visible. See the Standard for Formliner Details.

Designers are cautioned about introducing textures and relief on the inside faces of vehicle barriers. The degree of relief and texture can influence the vehicle response during a crash. See Chapter 30 – Railings for further guidance.

Ornamentation

If signs or medallions are necessary, refer to section 2-1-60 of the *Traffic Guideline Manual*.

Regarding ornamentation in general, more is seldom better.

"In bridge building… to overload a structure or any part thereof with ornaments… would be to suppress or disguise the main members and to exhibit an unbecoming wastefulness. The plain or elaborate character of an entire structure must not be contradicted by any of its parts."

- J.B. Johnson, 1912

4.5 Aesthetics Process

The structural design engineer needs to be involved early in the aesthetic decision making process. BOS should have early representation on projects with considerable aesthetic concerns. Throughout this process it is important to remember that aesthetics is a concept, not a commodity – it is about a look, not about what can be added to a structure.

WisDOT policy item:

For current statewide policy on aesthetic and/or decorative features (CSS), please see the *Program Management Manual* (PMM). See 4.3 for discussion on primary features such as shape and 4.9 for simple aesthetic concepts. The information below is current WisDOT policy. **Note: Any deviation from the standard details found in the WisDOT Bridge Manual regarding aesthetic features requires prior approval from BOS.**

Aesthetic and/or Decorative Items (non-Participating, or CSS Items)

- *All* formliner is considered CSS. This includes geometric patterns, vertical ribs, rock patterns, custom patterns/designs, etc.
- Stain
- Ornamentation, including city symbols, city names, etc.
- Fencing, railing, or parapets not described below.
- Structure shapes not defined in 4.3 and 4.9 or the standard details.

Note: Future maintenance costs can be substantial when factoring in not only surface preparation and stain/paint, but planning, mobilization and maintenance of traffic required that is entirely attributable to the maintenance project. For example, re-staining of concrete, when all project costs are accounted for, often exceeds $20/SF.

Participating (non-CSS) Items

- Street Names: Street names recessed in the bridge parapet, and stained for visibility, are considered a participating item. The street name is considered an assistance to drivers. Having the name in the parapet removes the sign from the side of the road, which is considered a maintenance problem and safety hazard.

- Protective Fence: Any standard fencing from the Wisconsin Bridge Manual is considered a participating item. Additional costs for decorative fencing requested by the municipality will be included as a non-participating item. Fencing can be either galvanized or a duplex system of galvanized with a colored polymer-coating and/or paint. The polymer coating and/or paint is a nominal cost that provides a longer service life for the fence.

- Bridge Rail: Any standard railing from the Wisconsin Bridge Manual is considered a participating item as long as the railing is required for pedestrian and/or bicyclist protection. There is no discernable difference in cost between any of the standard railings. Paint is a nominal cost that provides longer service life for the railing.

- Bridge Parapet: Any standard parapet from the Wisconsin Bridge Manual is considered a participating item. The Vertical Face Parapet 'TX' may be used as a participating item as long as the parapet is required for pedestrian and/or bicyclist protection. There is no discernable difference in cost between the Type 'TX' and a shorter, plain concrete parapet with railing that is often used for pedestrian and/or bicyclist protection.

4.6 Level of Aesthetics

The Regional Office should establish one of the following levels of aesthetics and indicate it on the Structure Survey Report. This will help the structural designer decide what level of effort and possible types of aesthetics treatments to consider. If Level 2 or greater is indicated, the Regional Office personnel or consultant must suggest particular requirements such as railing type, pier shape, special form liners, color, etc. in the comments area of the Structure Survey Report. Most Regions/municipalities prefer to leave anti-graffiti coating off of structures and would rather re-stain, as this is easier than trying to clean the graffiti.

Aesthetic treatments should be agreed upon prior to completion of preliminary plans in order for the final design to proceed efficiently. These details would be developed through the aesthetic process.

1. Level One: A general structure designed with standard structure details. This would apply in rural areas and urban areas with industrial development.

2. Level Two: Consists of cosmetic improvements to conventional Department structure types, such as the use of color stains/paints, texturing surfaces, modifications to fascia walls and beams or more pleasing shapes for columns. This would apply where there needs to be less visual impact from roadway structures.

3. Level Three: Emphasize full integration of efficiency, economy and elegance in structure components and the structure as a whole. Consider structure systems that are pleasing such as shaped piers and smooth superstructure lines. These structures would need to be in harmony with the surrounding buildings and/or the existing landscape.

4. Level Four: Provide overall aesthetics at the site with the structure incorporating level three requirements. The structure would need to blend with the surrounding terrain and landscaping treatment would be required to complete the appearance.

Note: The above text was left in this chapter, but will likely be modified or removed in future editions of this Manual. See 4.5 for current policy regarding CSS and levels of aesthetics.

4.7 Accent Lighting for Significant Bridges

The Wisconsin DOT will consider as part of an improvement project accent lighting for significant urban bridges with a clear span length of 450 feet and greater. The lighting would accent significant components above the driving surface such as an arch, truss, or a cable stayed superstructure. This lighting would enhance the noteworthy structure components of these significant bridges. The Traffic Guideline Manual (TGM) and the Highway Program Manual (HPM) have respective guidance of maintenance and cost share policy.

The following structures would fall into this definition of significant urban bridges:

"Name"	Region	County	Feature On	Feature Under	Year Built	Border
Tower Drive	NE	Brown	IH 43	Fox River	1979	
Praire du Chien	SW	Crawford	USH 18-STH 60	Mississippi River	1974	X
Blatnik	NW	Douglas	IH 535-USH 53	St Louis Bay	1961	X
Bong	NW	Douglas	USH 2	St Louis River	1983	X
Cass Arch	SW	La Crosse	USH 14 EB	Mississippi River	2004	X
Cass Truss	SW	La Crosse	USH 14 WB	Mississippi River	1940	X
Hoan Bridge	SE	Milwaukee	IH 794 WB-Lake Freeway	Milwaukee River	1974	
Dubuque (Iowa)	SW	Grant	USH 61-USH 151	Mississippi River	1982	X
Stillwater	NW	St Croix	TH 36	St Croix River	New	X

Table 4.4-1
Accent Lighting for Significant Bridges

4.8 Resources on Aesthetics

The Bridge Aesthetic Sourcebook from AASHTO is a very good source of practical ideas for short and medium span bridges. The Transportation Research Board (TRB) Subcommittee on Bridge Aesthetics authored this document and it can be found on the following website: The final printing of this guide (noted in the References) is available through the AASHTO publication website:

4.9 Non-CSS Aesthetic Concepts

Standards 4.02-4.05 provide details for acceptable non-CSS funded aesthetic concepts. The three types (Type I, Type II and Type III) show a plain wing, a wing with a rustication trim line and a wing with a recessed panel, respectively. For each given wing type, one or two acceptable parapet and/or pier details are shown.

- Type I: Simple features utilizing a plain wing, standard parapet and minimal pier rustications. Type I is ideal for most rural and some urban applications.

- Type II: The wings utilize the same rustication trim line as the columns. The columns can have single or paired rustication trim lines. Single rustication lines can be used for 32-inch parapets and double rustication lines can be used for 42-inch parapets. Type II can be used in urban applications and other limited areas.

- Type III: Recessed panel wings and recessed panel columns, along with standard parapets, are to be used in urban settings, only.

Within a given corridor, only one Type should be chosen so as not to create a disharmonious experience for those driving the corridor.

The following pages show renderings of the various non-CSS aesthetic concepts.

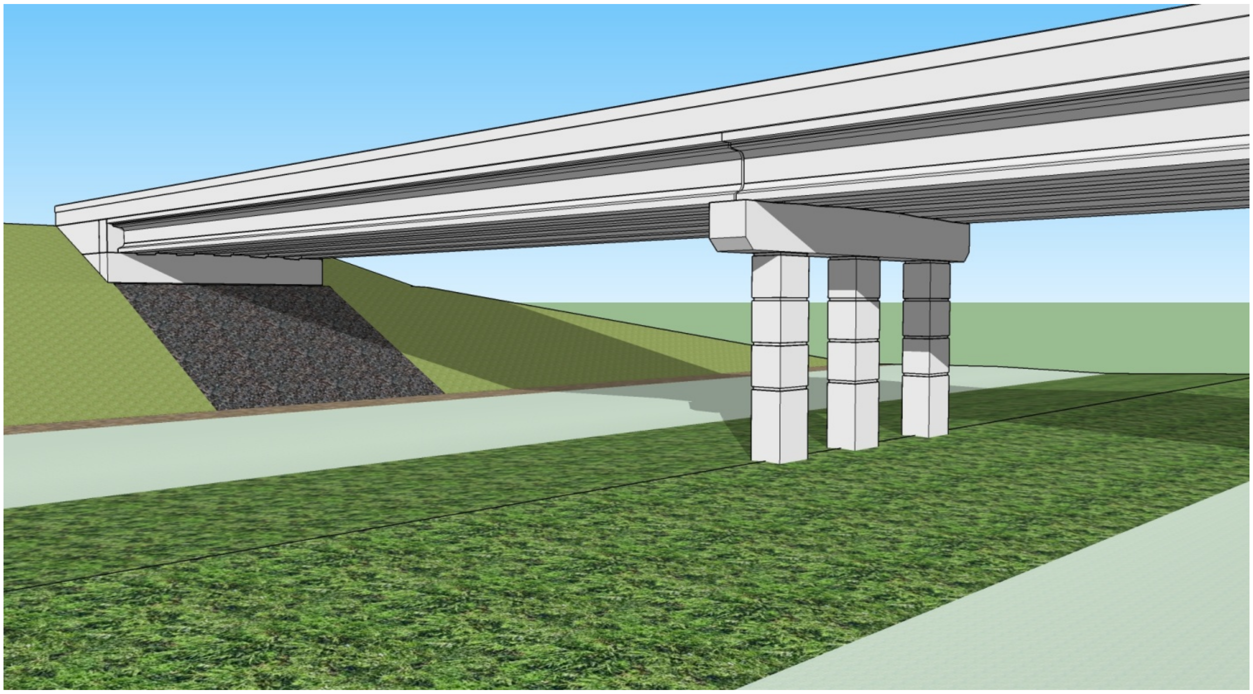

Figure 4.9-1
Aesthetic Concept Type I

- Plain abutment wings
- Single banded pier rustications
- Standard parapets
- Most rural and some urban applications

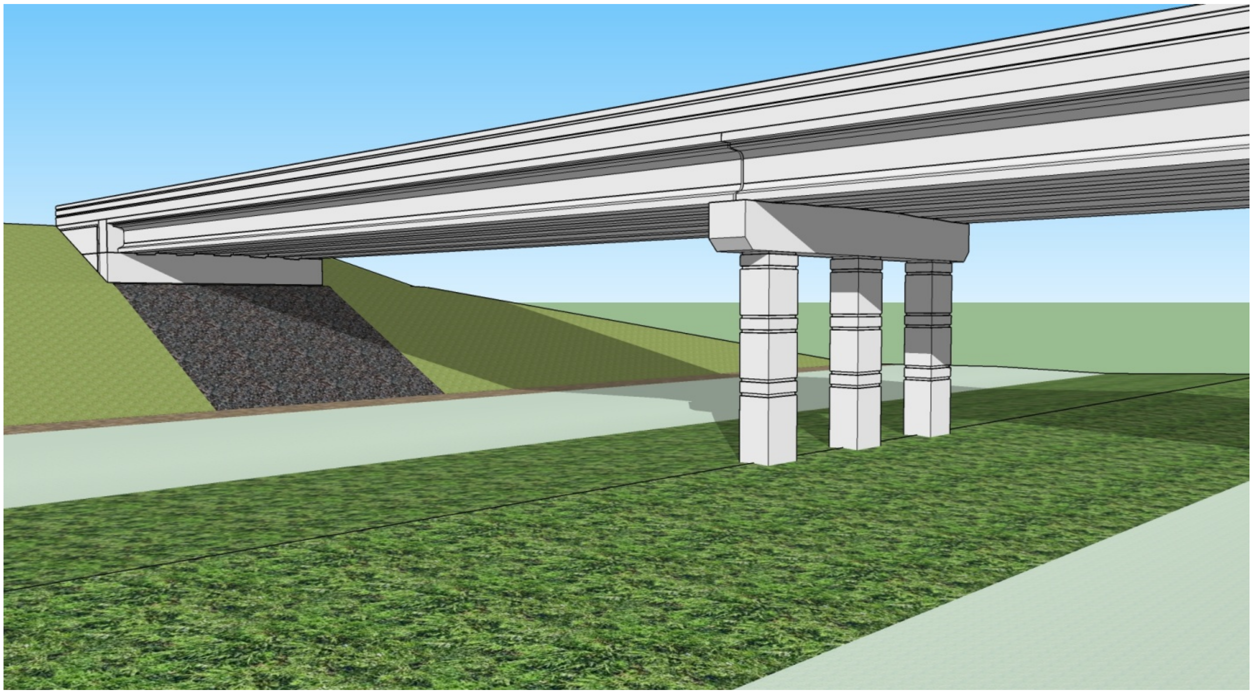

Figure 4.9-3
Aesthetic Concept Type II

- Rustication trim line on abutment wing
- Single or double banded pier rustications
- Rustication trim line(s) on parapets (one on 32” parapet and two on 42” parapet)
- Urban and other select applications

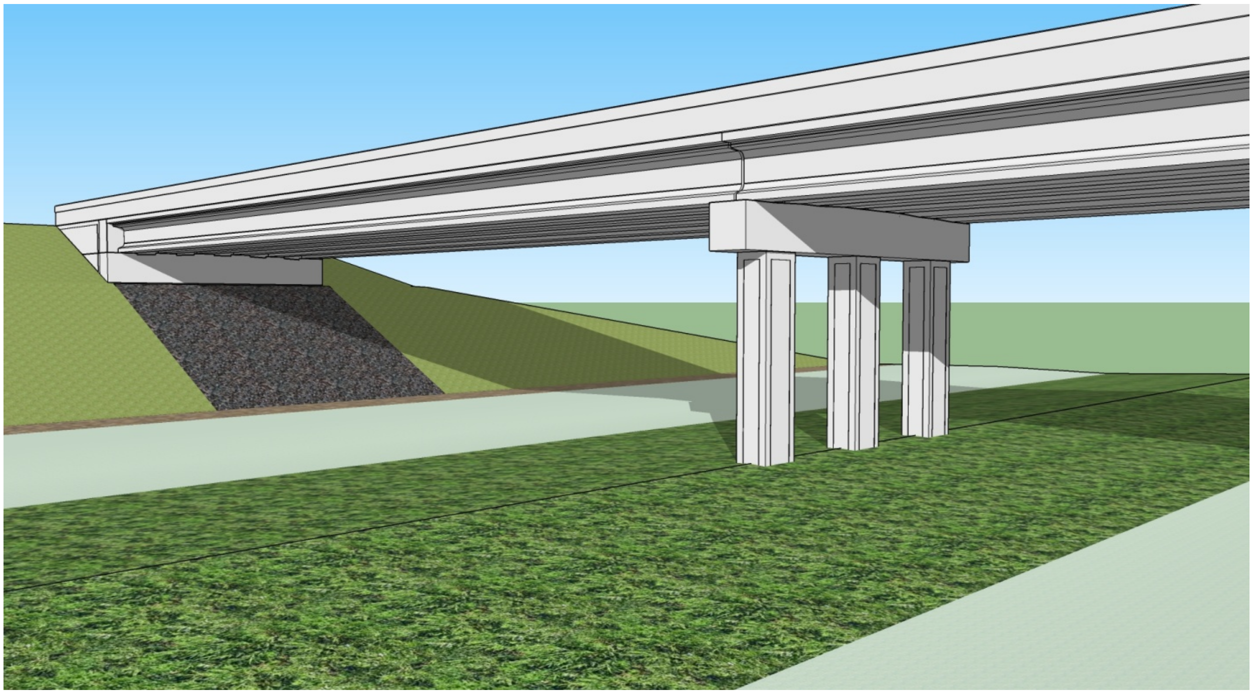

Figure 4.9-2
Aesthetic Concept Type III

- Recessed panel abutment wings
- Recessed panel columns
- Standard parapet
- Urban applications

4.10 References

1. AASHTO, *Bridge Aesthetics Sourcebook,* 2010.

2. Gottemoeller, Frederick, *Bridgescape: The Art of Designing Bridges*, John Wiley & Sons, Inc., 2004.

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5.1 Factors Governing Bridge Costs

Bridge costs are tabulated based on the bids received for all bridges let to contract. While these costs indicate some trends, they do not reflect all the factors that affect the final bridge cost. Each bridge has its own conditions which affect the cost at the time a contract is let. Some factors governing bridge costs are:

1. Location - rural or urban, or remote regions
2. Type of crossing
3. Type of superstructure
4. Skew of bridge
5. Bridge on horizontal curve
6. Type of foundation
7. Type and height of piers
8. Depth and velocity of water
9. Type of abutment
10. Ease of falsework erection
11. Need for special equipment
12. Need for maintaining traffic during construction
13. Limit on construction time
14. Complex forming costs and design details
15. Span arrangements, beam spacing, etc.

Figure 5.2-1 shows the economic span lengths of various type structures based on average conditions. Refer to Chapter 17 for discussion on selecting the type of superstructure.

Annual unit bridge costs are included in this chapter. The area of bridge is from back to back of abutments and out to out of the concrete superstructure. Costs are based only on the bridges let to contract during the period. In using these cost reports exercise care when a small number of bridges are reported as these costs may not be representative.

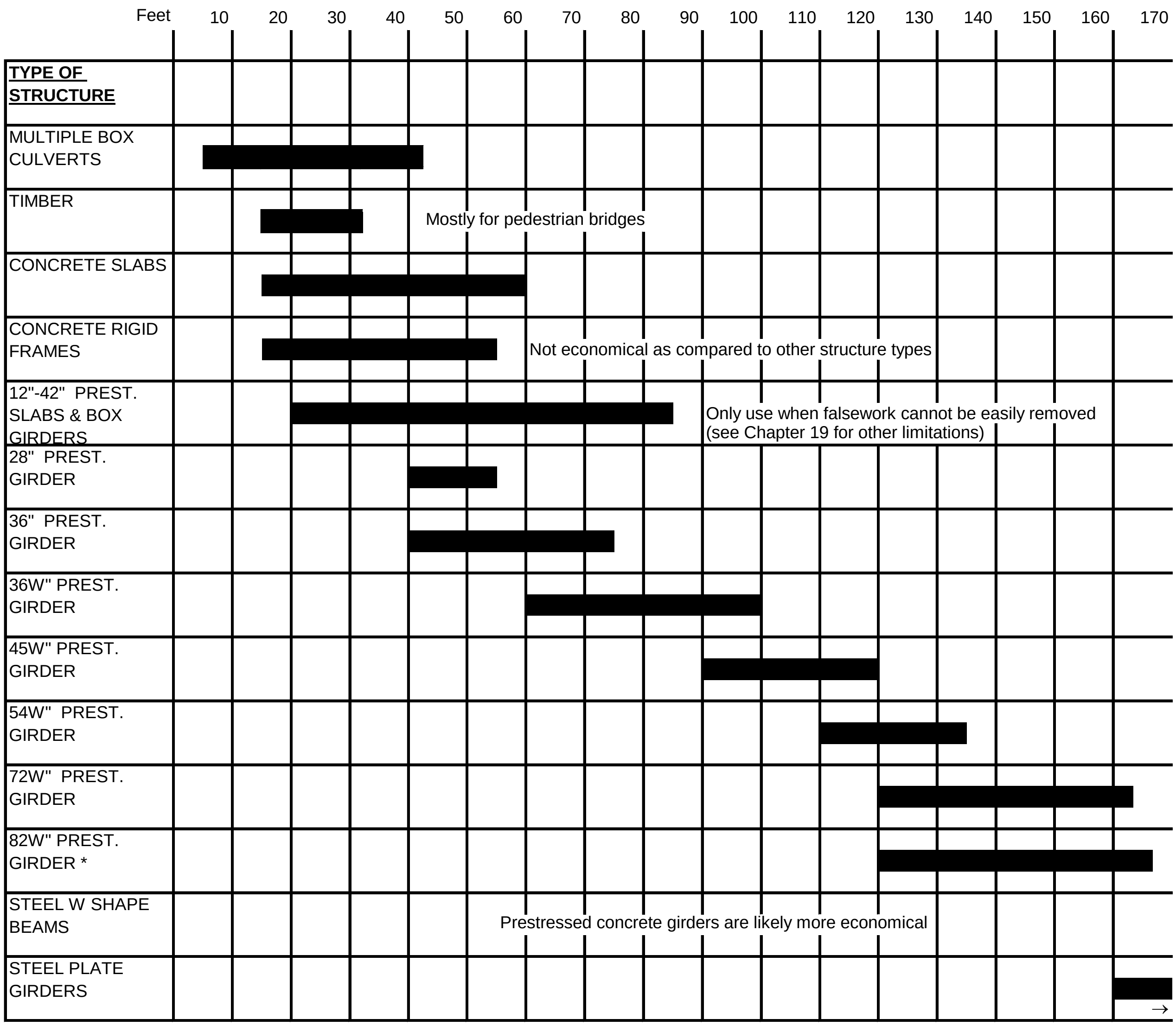
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

No costs are shown for rolled steel sections as these structures are not built very often. They have been replaced with prestressed girders which are usually more economical. The cost of plate girders is used to estimate rolled section costs.

For structures over a railroad, use the costs of grade separation structures. Costs vary considerably for railroad structures over a highway due to different railroad specifications.

Other available estimating tools such as *AASHTOWare Project Estimator* and *Bid Express*, as described in FDM 19-5-5, should be the primary tools for structure project cost estimations. Information in this chapter can be used as a supplemental tool.

5.2 Economic Span Lengths



*Currently there is a moratorium on the use of 82W" prestressed girders in Wisconsin
Note: Slab bridges should not be used on the Interstate

Figure 5.2-1
Economic Span Lengths

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
502.3100	Expansion Device (structure) (LS)	LF	210.97
502.3110.S	Expansion Device Modular (structure) (LS)	LF	969.95
SPV.0105	Expansion Device Modular LRFD (structure) (LS)	LF	1,947.75
513.2000	Railing Pipe (structure)	LF	137.02
513.4055	Railing Tubular Type H (structure) (LS)	LF	135.14
513.4060	Railing Tubular Type M (structure) (LS)	LF	228.70
513.4065	Railing Tubular Type PF (structure) (LS)	LF	179.96
513.4090	Railing Tubular Screening (structure) (LS)	LF	133.78
513.7005	Railing Steel Type C1 (structure) (LS)	LF	151.56
513.7010	Railing Steel Type C2 (structure) (LS)	LF	109.13
513.7015	Railing Steel Type C3 (structure) (LS)	LF	122.92
513.7020	Railing Steel Type C4 (structure) (LS)	LF	147.39
	Railing Steel Type C2 Pedestrian (structure) (LS)	LF	184.21
	Railing Steel Type C3 Pedestrian (structure) (LS)	LF	158.93
	Railing Steel Type C4 Pedestrian (structure) (LS)	LF	150.00
513.7050	Railing Type W (structure) (LS)	LF	120.30
513.7084	Railing Steel Type NY4 (structure)	LF	317.83

Table 5.3-1

Contract Unit Bid Prices for Structures - 2015

Other bid items should be looked up in Estimator or Bid Express

5.4 Bid Letting Cost Data

This section includes past information on bid letting costs per structure type. Values are presented by structure type and include: number of structures, total area, total cost, superstructure cost per square foot and total cost per square foot.

5.4.1 2011 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	36	218,311	18,719,353	50.45	85.75
Reinf. Conc. Slabs (All but A5)	22	63,846	7,135,430	52.90	111.76
Reinf. Conc. Slabs (A5 Abuts)	14	21,005	2,470,129	53.00	117.60

Table 5.4-1
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	44	337,346	31,596,585	65.90	93.66
Reinf. Conc. Slabs (All but A5)	6	33,787	3,462,995	52.90	102.49

Table 5.4-2
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	5	2,140.00
Twin Cell	6	1,998.00
Triple Cell	5	3,518.00
Precast	1	7,385.00

Table 5.4-3
Box Culverts

Railroad Bridge	Cost per Sq. Ft.
B-20-210	3,654.30

Table 5.4-4
Railroad Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	6	7,893	494,274	62.62
MSE Panel Walls	19	87,000	6,679,782	76.78
Concrete Walls	3	3,516	237,230	67.47
Panel Walls	2	14,832	3,458,722	233.19
Tangent Pile Walls	3	10,139	1,581,071	155.94
Wire Faced MSE Walls	18	149,735	11,412,474	76.22
Soldier Pile Walls	2	7,849	779,563	99.32

Table 5.4-5
Retaining Walls

5.4.2 2012 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	18	115,512	11,610,435	53.88	100.50
Reinf. Conc. Slabs (All but A5)	22	80,797	8,269,942	53.04	102.35
Reinf. Conc. Slabs (A5 Abuts)	3	6,438	739,983	53.24	114.95

Table 5.4-6
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	58	697,381	65,044,526	65.91	93.27
Reinf. Conc. Slabs (All but A5)	1	5,812	491,683	43.73	84.60

Table 5.4-7
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	5	1,516.50
Twin Cell	6	3,292.00
Triple Cell	5	2,624.60
Precast	1	--

Table 5.4-8
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
B-40-761/762	325.22

Table 5.4-9
Pre-Fabricated Pedestrian Bridges

Pedestrian Bridge	Cost per Sq. Ft.
B-53-265	91.93

Table 5.4-10
Pedestrian Bridges

Buried Slab Bridge	Cost per Sq. Ft.
C-13-155	170.77

Table 5.4-11
Buried Slab Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	17	30,536	1,604,280	52.54
MSE Panel Walls	25	111,365	7,215,980	64.80
Modular Walls	1	500	49,275	98.50
Concrete Walls	2	5,061	416,963	82.39
Panel Walls	2	6,476	1,094,638	169.03
Wire Faced MSE Walls	21	109,278	16,130,424	147.61
Secant Pile Walls	1	12,545	2,073,665	165.30
Soldier Pile Walls	2	4,450	298,547	66.49
MSE Gravity Walls	1	975	61,470	63.05
Steel Sheet Pile Walls	5	8,272	352,938	42.67

Table 5.4-12
Retaining Walls

5.4.3 2013 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	17	120,700	12,295,720	49.75	101.87
Reinf. Conc. Slabs (All but A5)	12	26,361	2,244,395	48.26	85.14
Reinf. Conc. Slabs (A5 Abuts)	5	8,899	992,966	49.28	111.58

Table 5.4-13
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	52	672,482	67,865,859	69.67	100.92
Steel Plate Girders	6	195,462	27,809,905	89.62	142.28
Trapezoidal Steel Box Girders	7	571,326	98,535,301	116.21	172.47

Table 5.4-14
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	11	1,853.00
Twin Cell	5	2,225.00
Precast	3	1,079.00

Table 5.4-15
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
B-13-666	240.30
B-17-211	174.33

Table 5.4-16
Pre-Fabricated Pedestrian Bridges

Pedestrian Bridge	Cost per Sq. Ft.
B-13-661	222.06
B-13-656	105.60
B-13-657	106.62
B-40-784	289.02

Table 5.4-17
Pedestrian Bridges

Buried Slab Bridge	Cost per Sq. Ft.
B-24-40	182.28
B-5-403	165.57
B-13-654	210.68

Table 5.4-18
Buried Slab Bridges

Railroad Bridge	Cost per Sq. Ft.
B-40-773	1,151.00
B-40-774	1,541.00

Table 5.4-19
Railroad Bridges

Inverted T Bridge	Cost per Sq. Ft.
B-13-608	192.75
B-13-609	235.01
B-40-89	528.81

Table 5.4-20
Inverted T Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	8	13,351	447,017	33.48
MSE Panel Walls	55	255,817	23,968,072	93.69
Concrete Walls	23	32,714	2,991,867	91.46
Panel Walls	7	39,495	8,028,652	203.28
Wired Faced MSE Walls	28	160,296	20,554,507	128.17

Table 5.4-21
Retaining Walls

5.4.4 2014 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	20	457,537	52,424,589	53.80	114.58
Reinf. Conc. Slabs (All but A5)	27	59,522	8,104,551	58.89	136.16
Reinf. Conc. Slabs (A5 Abuts)	9	16,909	2,150,609	56.13	127.19
Buried Slab Bridges	1	4,020	198,583	11.63	49.40

Table 5.4-22
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	29	409,929	44,335,036	64.66	108.15
Reinf. Conc. Slabs (All but A5)	2	15,072	1,739,440	47.68	115.41
Steel Plate Girders	3	85,715	15,669,789	114.08	182.81
Steel I-Beam	1	2,078	596,712	82.99	287.16
Pedestrian Bridges	3	35,591	7,436,429	--	208.94
Trapezoidal Steel Box Girders	1	59,128	9,007,289	121.00	152.34

Table 5.4-23
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	10	2,361.30
Twin Cell	4	2,584.21
Triple Cell	1	2,928.40
Triple Pipe	1	1,539.41

Table 5.4-24
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	11	13,856	755,911	54.55
MSE Panel Walls	36	319,463	23,964,444	75.01
Concrete Walls	7	58,238	8,604,747	147.75
Panel Walls	1	3,640	590,682	162.28
Wired Faced MSE Walls	2	3,747	537,173	143.36
Secant Pile Walls	1	68,326	7,488,658	109.60
Soldier Pile Walls	9	33,927	4,470,908	131.78
Steel Sheet Pile Walls	2	3,495	159,798	45.72

Table 5.4-25
Retaining Walls

Noise Walls	Total Area (Sq. Ft)	Total Costs	Cost per Sq. Ft.
13	200,750	5,542,533	27.61

Table 5.4-26
Noise Walls

5.4.5 2015 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	22	338,229	41,220,154	60.96	121.87
Reinf. Conc. Slabs (Flat)	26	47,766	7,151,136	62.77	149.71
Reinf. Conc. Slabs (Haunched)	6	27,967	3,517,913	57.49	125.79
Buried Slab Bridges	1	2,610	401,000	43.74	153.64
Pre-Fab Pedestrian Bridges	3	29,304	3,440,091	--	117.39

Table 5.4-27
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	58	768,458	102,067,913	66.04	132.82
Reinf. Conc. Slabs (Flat)	2	8,566	922,866	46.36	107.74
Reinf. Conc. Slabs (Haunched)	1	6,484	868,845	41.26	133.99
Steel Plate Girders	4	100,589	20,248,653	137.13	201.30
Trapezoidal Steel Box Girders	4	305,812	79,580,033	189.24	260.23
Rigid Frames	2	7,657	2,730,308	--	356.58
Timber	1	16,800	1,982,669	--	118.02
Pre-Fab Pedestrian Bridges	1	1,851	449,475	--	242.83

Table 5.4-28
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	2	2,235.67
Twin Cell	6	3,913.05
Single Pipe	1	2,262.11
Twin Pipe	2	426.20
Triple Pipe	2	1,424.09
Quadruple Pipe	1	2,332.96

Table 5.4-29
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	11	22,353	1,594,171	71.32
MSE Panel Walls	51	315,440	28,038,238	88.89
Wire Faced MSE Walls	3	10,345	1,501,948	145.19
Wired Faced MSE Walls w/ Precast Conc. Wall Panels	12	50,670	10,195,161	201.21
Secant Pile Walls	1	5,796.50	1,075,785	185.59
Soldier Pile Walls	6	37,498	6,037,788	161.02
Steel Sheet Pile Walls	6	11,319	668,227	59.04
Concrete Walls	2	6,850	712,085	103.96
MSE Panel Walls w/Integral Barrier	4	14,330	1,098,649	76.67

Table 5.4-30
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	2	44	122,565	2,785.56
	1-Steel Col.	2	42	63,965	1,522.98
Butterfly (2-Signs)	1-Steel Col.	1	21	48,971	2,331.97
Cantilever	Conc. Col	18	530	1,217,454	2,297.08
	1-Steel Col.	15	394	528,950	1,342.85
Full Span	Conc. Col.	44	4,035	5,309,906	1,315.96
	1-Steel Col.	12	720	476,598	662.00
	2-Steel Col.	10	711	775,858	1,091.22
Full Span + Cantilever	Conc. Col.	1	84	166,003	1,976.22

Table 5.4-31
Sign Structures

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6.1 Approvals, Distribution and Work Flow

Production of Structural Plans

Regional Office	Prepare Structure Survey Report.
Geotechnical Section (Bur. of Tech. Services)	Make site investigation and prepare Site Investigation Report. See 6.2.1 for exceptions.
Structures Development Sect. (BOS)	Record Structure Survey Report.
Structures Design Section (BOS)	Determine type of structure.
	Perform hydraulic analysis if required.
	Check roadway geometrics and vertical clearance.
	Review Site Investigation Report and determine foundation requirements. Develop scour computations for bridges and record scour code on the preliminary plans.
	Draft preliminary plan layout of structure.
	Send copies of preliminary plans to Regional Office.
	If Federal aid funding is involved, send copies of preliminary plans to the Federal Highway Administration for major, moveable, and unusual bridges.
	If a waterbody that qualifies as a “navigable water of the United States” is crossed, a Permit drawing to construct the bridge is sent to the Coast Guard. If FHWA determines that a Coast Guard permit is needed, send a Permit drawing to the Coast Guard. If Federal aid is involved, preliminary plans are sent to

	the Federal Highway Administration for approval.
	Review Regional Office comments and other agency comments, modify preliminary plans as necessary.
	Review and record project for final structural plan preparation.
Structures Design Units (BOS)	Prior to starting project, Designer contacts Regional Office to verify preliminary structure geometry, alignment, width and the presence of utilities.
	Prepare and complete plans, specs and estimates for the specified structure.
	Give completed job to the Supervisor of Structures Design Unit.
Supervisor, Structures Design Unit (BOS)	Review plans, specs and estimates.
	Send copies of final structural plans and special provisions to Regional Offices.
	Sign lead structural plan sheet.
	Deliver final structural plans and special provisions to the Bureau of Project Development.
Bur. of Project Development	Prepare final approved structural plans for pre-contract administration.

See FDM Section 21-30-1.3 for information on determining whether a bridge crossing falls under the Coast Guard's jurisdiction.

6.2 Preliminary Plans

6.2.1 Structure Survey Report

The Structure Survey Report is prepared by Regional Office or consultant personnel to request a structure improvement project. The following forms in word format are used and are available at: http://www.dot.wisconsin.gov/forms/index.htm

Under the "Plans and Projects" heading:

DT1694	Separation Structure Survey Report
DT1696	Rehabilitation Structure Survey Report
DT1698	Stream Crossing Structure Survey Report (use for Culverts also)

The front of the form lists the supplemental information to be included with the report.

6.2.1.1 BOS-Designed Structures

When preparing the Structure Survey Report, the region or consultant roadway designers will make their best estimate of structure type and location of substructure units. The completed Structure Survey Report with the locations of the substructure units and all required attachments and supporting information will then be submitted to the Bureau of Structures via e-submit (as "BOS Design") and also to the Geotechnical Section, through the Regional Soils Engineer. This submittal will take place a minimum of 18 months in advance of the earliest PS&E due date shown on the Structure Survey Report. The Geotechnical Section is responsible for scheduling and conducting the necessary soil borings. The Bureau of Structures and the Geotechnical Section will coordinate activities to deliver the completed structure plans on schedule.

When a geotechnical consultant is performing the subsurface exploration, the work typically proceeds after the preliminary plans have been assembled by the Bureau of Structures. Under some circumstances, it may be expected that the geotechnical information gathered will be included in the Structure Survey Report in advance of the development of the preliminary plans. In the case of the Geotechnical Section performing the subsurface exploration, the geotechnical work will proceed after the preliminary plans have been assembled by the Bureau of Structures.

The Project Manager may request information on structure type and substructure locations from the Bureau of Structures if such information is necessary to expedite the environmental process.

Under this process, the scheduling of geotechnical work is coordinated with the Bureau of Structures toward completion of the bridge plans by the final plan due date. If other geotechnical work is required for the project, the Project Manager should coordinate with the

Regional Soils Engineer and the Geotechnical Section to promote efficiency of field drilling operations.

If the preliminary plans are required more than one year in advance of the final plan due date due to the unique needs of the project, the Project Manager should discuss this situation with the Bureau of Structures Design Supervisor prior to submitting the Structure Survey Report.

Coordination early in the design process with DNR regarding removal techniques for the existing structure (if applicable), and new structure placement and type is very important. The status of any agreements with the DNR, which affect the structure, should be noted under additional information on the Structure Survey Report.

6.2.1.2 Consultant-Designed Structures

When preparing the Structure Survey Report, the region or consultant roadway designer's responsibility for submitting the Structure Survey Report depends on their involvement with the design of the structure and the soils investigation. Refer to Table 30.1 in FDM 3-20-30.2.2 for the process involved with differing levels of involvement.

If the preliminary bridge plans are required more than one year in advance of the final plan due to the unique needs of the project, the Project Manager should discuss this situation with the consultant.

Coordination early in the design process with DNR regarding removal techniques for the existing structure (if applicable), and new structure placement and type is very important. The status of any agreements with the DNR, which affect the structure, should be noted under additional information on the Structure Survey Report.

6.2.2 Preliminary Layout

6.2.2.1 General

The preparation of a preliminary layout for structures is primarily for the purpose of presenting an exhibit to the agencies involved for approval, before proceeding with final design and preparation of detail plans. When all the required approvals are obtained, the preliminary layout is used as a guide for final design and plan preparation.

The drawings for preliminary layouts are on sheets having an overall width of 11 inches and an overall length of 17 inches and should be placed within the current sheet border under the #8 tab.

6.2.2.2 Basic Considerations

The following criteria are used for the preparation of preliminary plans.

1. Selection of Structure Type. Refer to Chapter 17 - Superstructure-General, for a discussion of structure types.

2. Span Arrangements. For stream crossings the desired minimum vertical clearance from high water to low chord is given in Chapter 8 - Hydraulics. Span lengths for multiple span stream crossings are in most cases a matter of economics and the provision for an opening that adequately passes flood flows, ice and debris. For structures over waterways that qualify as navigable waters of the United States, the minimum vertical and horizontal clearances of the navigable span are determined by the U.S. Coast Guard after considering the interests of both highway and waterway transportation users.

   For most of the ordinary grade separation structures the requirements for horizontal clearance determine the span arrangements. Refer to Chapter 17 - Superstructure-General for span length criteria.

3. Economics.

   Economy is a primary consideration in determining the type of structure to be used. Refer to Chapter 5 – Economics and Costs, for cost data.

   At some stream crossings where the grade line permits considerable head room, investigate the economy of a concrete box culvert versus a bridge type structure. When economy is not a factor, the box culvert is the preferred type from the standpoint of maintenance costs, highway safety, flexibility for roadway construction, and provision of a facility without roadway width restrictions.

4. Aesthetics. Recognition of aesthetics as an integral part of a structure is essential if bridges are to be designed in harmony with adjacent land use and development. Refer to Chapter 4 - Aesthetics.

5. Hydraulic Consideration. Stream crossing structures are influenced by stream flow, drift, scour, channel conditions, ice, navigation, and conservation requirements. This information is submitted as part of the Structure Survey Report. Refer to Chapter 8 - Hydraulics for Hydraulic considerations and Section 8.1.5 for Temporary Structure Criteria.

6. Geometrics of Design. The vertical and horizontal clearance roadway widths, design live loading, alignment, and other pertinent geometric requirements are given in Chapter 3.

7. Maintenance. All bridge types require structural maintenance during their service life. Maintenance of approaches, embankments, drainage, substructure, concrete deck, and minor facilities is the same for the various types of bridges. A minimum draining grade of 0.5% across the bridge is desirable to eliminate small ponds on the deck except for open railings where the cross slope is adequate.

   Epoxy coated bar steel is required in all new decks and slabs.

   Steel girders require periodic painting unless a type of weathering steel is used. Even this steel may require painting near the joints. It is more difficult to repaint steel girders that span busy highways.

Cast-in-place reinforced concrete box girders and voided slabs have a poor experience in Wisconsin. They should not be used on new structures.

Deck expansion joints have proved to be a source of maintenance problems. Bridges designed with a limited number of watertight expansion devices are recommended.

8. Construction. Occasionally a structure is proposed over an existing highway on which traffic must be maintained. If the roadway underneath carries high volumes of traffic, any obstruction such as falsework would be hazardous as well as placing undesirable vertical clearance restrictions on the traveled way. This is also true for structures over a railroad.

   For structures over most high-volume roadways construction time, future maintenance requirements, and provision for future expansion of the roadway width, have considerable influence on the selection of the final product.

9. Foundations. Poor foundation conditions may influence the structure geometry. It may be more economical to use longer spans and fewer substructure units or a longer structure to avoid high approach fills.

10. Environmental Considerations. In addition to the criteria listed above all highway structures must blend with the existing site conditions in a manner that is not detrimental to environmental factors. Preservation of fish and wildlife, pollution of waters, and the effects on surrounding property are of primary concern in protecting the environment. The design of structures and the treatment of embankments must consider these factors.

11. Safety. Safety is a prime consideration for all aspects of the structure design and layout. Bridge railings are approved through actual vehicle crash testing.

6.2.2.3 Requirements of Drawing

6.2.2.3.1 Plan View

The plan view is preferably placed in the upper left-hand portion of the sheet at the largest scale practical (1”=10’) and shows the following basic information:

1. The plan view shall be shown with the reference line stationing progressing upstation from left to right on the sheet. A reference north arrow shall be included.

2. Structure span lengths, (center-to-center of piers and to centerline of bearing at abutments, end distance from centerline of bearing to back face of abutment and overall length of structure).

3. Dimensions along the reference line except for structures on a curve in which case they are along a tangent to the curve.

4. Stations are required at centerline of piers, centerline of bearing at abutments, and end of deck or slab.

5. Stations at intersection with reference line of roadway underneath for grade separation structures.

6. Direction of stationing increase for highway or railroad beneath a structure.

7. Detail the extent of slope paving or riprap.

8. Direction of stream flow and name if a stream crossing.

9. Highway number and direction and number of traffic lanes.

10. Horizontal clearance dimensions, pavement, shoulder, sidewalk, and structure roadway widths.

11. Median width if dual highway.

12. Skew angles and angles of intersection with other highways, streets or railroads.

13. Horizontal curve data if within the limits of the structure showing station of P.C., P.T., and P.I. Complete curve data of all horizontal curves which may influence layout of structure.

14. Location and dimension of minimum vertical clearance for highway or railroad grade separation structures.

    a. The minimum vertical clearance should be noted as the “Point of Minimum Vertical Clearance” for all spans.

    b. Minimum vertical clearance is required for each span of a structure above a traveled way (i.e., roadway, railroad track, etc.).

    c. Refer to *Facilities Development Manual 11-35-1, Section 1.5* for guidance pertaining to the required locations to be checked for underclearance.

15. If floor drains are proposed the type, approximate spacing, and whether downspouts are to be used.

16. Existing structure description, number, station at each end, buildings, underground utilities and pole lines giving owner's name and whether to remain in place, be relocated or abandoned.

17. Indicate which wingwalls have beam guard rail attached if any and wing lengths.

18. Structure numbers on plan.

19. Excavation protection for railroads.

20. Limits of railroad right-of-way. The locations are for reference only and need not be dimensioned.

21. Location of deck lighting or utilities if any.

22. Name Plate location.

23. Bench Mark Cap Location

24. Locations of surface drains on approach pavement.

25. Tangent offsets between reference line and tangent line along C_L substructure unit. Also include tangent offsets for edge of deck and reference line at 10 foot intervals.

6.2.2.3.2 Elevation View

The elevation view is preferably placed below the plan view. If the structure is not skewed the substructure units are to be a straight projection from the plan view. If skewed, the elevation is a view normal to substructure units. The view shows the following basic information:

1. Profile of existing groundline or streambed.

2. Cross-section of highway or channel below showing back slopes at abutments.

3. Elevation of top of berm and rate of back slope used in figuring length of structure.

4. Type and extent of slope paving or riprap on back slopes.

5. Proposed elevations of bottom of footings and type of piling if required.

6. Depth of footings for piers of stream crossing and if a seal is required, show and indicate by a note.

7. Location and dimension of minimum vertical clearance.

    a. Minimum vertical clearance is required for each span of a structure above a traveled way (i.e., roadway, railroad track, etc.).

    b. Refer to *Facilities Development Manual 11-35-1, Section 1.5* for guidance pertaining to the required locations to be checked for underclearance.

8. Streambed, observed and high water elevations for stream crossings.

9. Location of underground utilities, with size, kind of material and elevation indicated.

10. Location of fixed and expansion bearings.

11. Location and type of expansion devices.

12. Limits of railroad right-of-way. The locations are for reference only and need not be dimensioned.

An elevation view is required for deck replacements, overlays with full-depth deck repair and painting plans (or any rehabilitation requiring the contractor to go beneath the bridge). Enough detail should be given to provide the contractor an understanding of what is beneath the bridge (e.g. roadway, bike path, stream, type of slope paving, etc.).

6.2.2.3.3 Cross-Section View

A view of a typical pier is shown as a part of the cross-section. The view shows the following general information:

1. Slab thickness, curb height and width, type of railing.
2. Horizontal dimensions tied into a reference line or centerline of roadway.
3. Girder spacing with girder depth.
4. Direction and amount of crown or superelevation, given in %.
5. Point referred to on profile grade.
6. Type of pier with size and number of columns proposed.
7. For solid, hammerhead or other type pier approximate size to scale.
8. Dimension minimum depth of bottom of footings below ditch or finished ground line or if railroad crossing below top of rail.
9. Location for public and private utilities to be carried in the superstructure. Label owner's name of utilities.
10. Location of lighting on the deck or under the deck if any.

6.2.2.3.4 Other Requirements

1. Profile grade line across structure showing tangent grades and length of vertical curve. Station and elevation of P.C., P.I., P.T., and centerline of all substructure units.

   Profile grade line of highway beneath structure if highway separation or of top rail if railroad separation. Stations along top of rail are to be tied into actual stationing as established by the railroad company.

2. Channel change section if applicable. Approximate stream bed elevation at low point.
3. Any other view or detail which may influence the bridge type, length or clearance.
4. List design data including:

   Material Properties:

- Concrete Superstructure
- Concrete Substructure
- Bar Steel Reinforcement
- Structural Steel
- Prestressed Concrete
- Prestressing Steel

*Note: For rehabilitation projects, include Material Properties only for those materials utilized in the rehabilitation.

Foundations

- Soil Bearing Pressure
- Pile Type and Capacity (see 6.3.2.1)

Ratings (Plans Including Ratings that have been changed)

Live Load:

Design Loading: HL-93

Inventory Rating Factor: RF = X.XX

Operating Rating Factor: RF = X.XX

Wisconsin Standard Permit Vehicle (Wis-SPV) = XXX kips

(See Chapter 45 – Bridge Rating (45.8.2) for additional information)

Ratings (Plans Including Ratings that have not been changed)

Live Load:

Design Loading: HL-93 (taken from HSI, xx/xx/2xxx)

Inventory Rating Factor: RF = X.XX (taken from HSI, xx/xx/2xxx)

Operating Rating Factor: RF = X.XX (taken from HSI, xx/xx/2xxx)

Wisconsin Standard Permit Vehicle (Wis-SPV) = XXX kips (taken from HSI, xx/xx/2xxx)

If widening a bridge, provide ratings for both the new and existing superstructure elements. For example, if widening a girder bridge previously designed with Load

Factor Design, provide the LFR rating for the controlling existing girder and the LRFR rating for the controlling new girder.

Hydraulic Data

100 YEAR FREQUENCY

Q_{100} = XXXX C.F.S.
VEL. = X.X F.P.S.
HW_{100} = EL. XXX.XX
WATERWAY AREA = XXX SQ.FT.
DRAINAGE AREA = XX.X SQ.MI.
ROADWAY OVERTOPPING = (NA or add "Roadway Overtopping Frequency" data)
SCOUR CRITICAL CODE = X

2 YEAR FREQUENCY

Q_2 = XXXX C.F.S.
VEL. = X.X F.P.S.
HW_2 = EL. XXX.XX

ROAD OVERTOPPING FREQUENCY (if applicable, frequencies < 100 years)

FREQUENCY = XX YEARS
Q_{XX} = XXXX C.F.S.
HW_{XX}= EL. XXX.XX

(See Chapter 8 – Hydraulics for additional information)

5. Show traffic data. Give traffic count, data and highway for each highway on grade separation or interchange structure.

6.2.2.4 Utilities

In urban areas, public and private utilities generally have their facilities such as sewers, water cables, pipes, ducts, etc., underground, or at river crossings, in the streambed.

If these facilities cannot be relocated, they may interfere with the most economical span arrangement. The preferred location of light poles is at the abutments or piers.

Overhead power lines may cause construction problems or maintenance inspection problems. Verify if they exist and notify Utilities & Access Management Unit (Bureau of Tech. Services) to have them removed.

It is the general policy to not place utilities on the structure. The Utilities & Access Management Unit approves all utility applications and determines whether utilities are placed on the structures or can be accommodated some other way. Refer all requests to them. Also see Chapter 18 of the FDM and Chapter 4 of "*WisDOT Guide to Utility Coordination*".

6.2.3 Distribution of Exhibits

6.2.3.1 Federal Highway Administration (FHWA).

FHWA memorandums “Implementing Guidance-Project Oversight under Section 1305 of the Transportation Equity Act for the 21st Century (TEA-21) of 1998” dated August 20, 1998, and “Project Oversight Unusual Bridges and Structures” dated November 13, 1998, indicate that **FHWA Headquarters Bridge Division or the Division Office must review and approve preliminary plans for unusual bridges and structures on the following projects:**

1. Projects on the Interstate System
2. Projects on the National Highway System (NHS) but not on the Interstate System, unless it is determined by FHWA and WisDOT that the responsibilities can be assumed by WisDOT
3. Projects on non-NHS Federal-aid highways, and eligible projects on public roads which are not Federal-aid highways if WisDOT determines that it is not appropriate for WisDOT to assume the responsibilities

Technical assistance is also available upon request for projects/structures that are not otherwise subject to FHWA oversight.

Unusual bridges have the following characteristics:

- Difficult or unique foundation problems
- New or complex designs with unique operational or design features
- Exceptionally long spans
- Design procedures that depart from currently recognized acceptable practices

Examples of unusual bridges:

- Cable-stayed
- Suspension
- Arch
- Segmental concrete
- Movable
- Truss
- Bridge types that deviate from AASHTO bridge design standards or AASHTO guide specifications for highway bridges

- Major bridges using load and resistance factor design specifications
- Bridges using a three-dimensional computer analysis
- Bridges with spans exceeding 350 feet

Examples of unusual structures:

- Tunnels
- Geotechnical structures featuring new or complex wall systems or ground improvement systems
- Hydraulic structures that involve complex stream stability countermeasures, or designs or design techniques that are atypical or unique

Timing of submittals is an important consideration for FHWA approval and assistance, therefore, **FHWA should be involved as early as possible.**

The following preliminary documents should be submitted electronically (PDF format) to FHWA:

1. Preliminary plans (Type, Size and Location)
2. Bridge/structures related environmental concerns and suggested mitigation measures
3. Studies of bridge types and span arrangements
4. Approach bridge span layout plans and profile sheets
5. Controlling vertical and horizontal clearance requirements
6. Roadway geometry
7. Design specifications used
8. Special design criteria
9. Cost estimates
10. Hydraulic and scour design studies/reports showing scour predictions and related mitigation measures
11. Geotechnical studies/reports
12. Information on substructure and foundation types

Note: Much of this information may be covered by the submittal of a Structure Type Selection Report.

6.2.3.2 Other Agencies

This is a list of other agencies that may or may not need to be coordinated with. There may be other stakeholders that require coordination. Consult Chapter 5 of the Facilities Development Manual (FDM) for more details on coordination requirements.

- Department of Natural Resources

  A copy of preliminary plans (preliminary layout, plan & profile, and contour map) for stream crossing bridges are forwarded by BOS to the Department of Natural Resources for comment, in accordance with the cooperative agreement between the Department of Transportation and the Department of Natural Resources. (See Chapter 8 - Hydraulics).

- Railroad (FDM Chapter 17)

  Begin communicating as early as possible with the Region Railroad Coordinator.

- Utilities (FDM Chapter 18, Bridge Manual Chapter 32)

  BOS discourages attachment of utilities to a structure. However, if there are no other viable options, private or public utilities desiring to attach their facilities (water, and sewer mains, ducts, cables, etc.) to the structure must apply to the owner for approval. For WisDOT owned structures, approval is required from the Region's Utilities & Access Management Unit.

- Coast Guard (FDM)

- Regions

  A copy of the preliminary plans is sent to the Regional Office involved for their review and use.

- Native American Tribal Governments

- Corps of Engineers

- Other governing municipalities

- State Historic Preservation Office

- Environmental Protection Agency

- Other DOTs

6.3 Final Plans

This section describes the general requirements for the preparation of construction plans for bridges, culverts, retaining walls and other related highway structures. It provides a standard procedure, form, and arrangement of the plans for uniformity.

6.3.1 General Requirements

6.3.1.1 Drawing Size

Sheets are 11 inches wide from top to bottom and 17 inches long. A border line is provided on the sheet 1 inch from the left edge, and ¼ inch from other edges. Title blocks are provided on the first sheet for a signature and other required information. The following sheets contain the same information without provision for a signature.

6.3.1.2 Scale

All drawings insofar as possible are drawn to scale. Such details as reinforcing steel, steel plate thicknesses, etc. are not scaled. The scale is adequate to show all necessary details.

6.3.1.3 Line Thickness

Object lines are the widest line on the drawing. Lines showing all or part of an existing structure or facility are shown by dashed lines of somewhat lighter weight.

Lines showing bar steel are lighter than object lines and are drawn continuous without any break. Dimension and extension lines are lighter than bar steel lines but heavy enough to make a good reproduction.

6.3.1.4 Lettering and Dimensions

All lettering is upper case. Lettering and dimensions are read from the bottom or right hand side and should be placed above the dimension lines. Notes and dimension text are 0.12 inches high; view titles are 0.20 inches high (based on full size sheet, 22” x 34”). Dimensions are given in feet and inches. Elevations are given in decimal form to the nearest 0.01 of a foot. Always show two decimal places. Although plan dimensions are very accurate, the contractor should use reasonable tolerances during construction of the project by building to the accuracy required. Detail structural steel to the thickness of the material involved.

6.3.1.5 Notes

Show any notes to make the required details clear on the plans. Do not include material that is part of the specifications.

6.3.1.6 Standard Insert Drawings

Standard detail sheets are available for railings and parapets, prestressed girders, bearings, expansion joints, and drains. Fill in the dimensions and titles required and insert in the final plans.

Standard insert sheets can be found at: http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/insert-sheets.aspx

6.3.1.7 Abbreviations

Abbreviations are to be used throughout the plans whenever possible. Abbreviations approved to be used are as follows:

Abutment	ABUT.	East	E.
Adjacent	ADJ.	Elevation	EL.
Alternate	ALT.	Estimated	EST.
And	&	Excavation	EXC.
Approximate	APPROX.	Expansion	EXP.
At	@	Fixed	F.
Back Face	B.F.	Flange Plate	Fl. Pl.
Base Line	B/L	Front Face	F.F.
Bench Mark	B.M.	Galvanized	GALV.
Bearing	BRG.	Gauge	GA.
Bituminous	BIT.	Girder	GIR.
Cast-in-Place	C.I.P.	Highway	HWY.
Centers	CTRS.	Horizontal	HORIZ.
Center Line	C/L	Inclusive	INCL.
Center to Center	C to C	Inlet	INL.
Column	COL.	Invert	INV.
Concrete	CONC.	Left	LT.
Construction	CONST.	Left Hand Forward	L.H.F.
Continuous	CONT.	Length of Curve	L.
Corrugated Metal Culvert Pipe	C.M.C.P.	Live Load	L.L.
Cross Section	X-SEC.	Longitudinal	LONGIT.
Dead Load	D.L.	Maximum	MAX.
Degree of Curve	D.	Minimum	MIN.
Degree	°	Miscellaneous	MISC.
Diaphragm	DIAPH.	North	N.
Diameter	DIA.	Number	NO.
Discharge	DISCH.	Near Side, Far Side	N.S.F.S.
Per Cent	%	Sidewalk	SDWK.
Plate	PL	South	S.
Point of Curvature	P.C.	Space	SPA.
Point of Intersection	P.I.	Specification	SPEC

Point of Tangency	P.T.	Standard	STD.
Point on Curvature	P.O.C.	Station	STA.
Point on Tangent	P.O.T.	Structural	STR.
Property Line	P.L.	Substructure	SUBST.
Quantity	QUAN.	Superstructure	SUPER.
Radius	R.	Surface	SURF.
Railroad	R.R.	Superelevation	S.E.
Railway	RY.	Symmetrical	SYM
Reference	REF.	Tangent Line	TAN. LN.
Reinforcement	REINF.	Transit Line	T/L
Reinforced Concrete Culvert Pipe	R.C.C.P.	Transverse	TRAN.
Required	REQ'D.	Variable	VAR.
Right	RT.	Vertical	VERT.
Right Hand Forward	R.H.F.	Vertical Curve	V.C.
Right of Way	R/W	Volume	VOL.
Roadway	RDWY.	West	W.
Round	ø	Zinc Gauge	ZN. GA.
Section	SEC.		

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

Universally accepted nomenclature and approved definitions are to be used wherever possible.

6.3.2 Plan Sheets

The following information describes the order of plan sheets and the material required on each sheet.

Plan sheets are placed in order of construction generally as follows:

1. General Plan
2. Subsurface Exploration
3. Abutments
4. Piers
5. Superstructure and Superstructure Details
6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

See the BOS web page, CADD Resource Files, for the latest sheet borders to be used. Sheet borders are given for new bridges, rehabilitation projects and concrete box culverts. A superstructure replacement utilizing the existing substructure, bridge widenings, as well as damaged girder replacements should use the sheet border for a new structure. See Chapter 40 - Bridge Rehabilitation for criteria as to when superstructure replacements are allowed.

1. Plan View

   Same requirements as specified for preliminary drawing, except do not show contours of groundline and as noted below.

   a. Sufficient dimensions to layout structure in the field.

   b. Describe the structure with a simple note such as: Four span continuous steel girder structure.

   c. Station at end of deck on each end of bridge.

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

   a. For railroad bridges show a railroad cross-section.

   b. View of pier if the bridge has a pier (s), if not, view of abutment.

4. Grade Line

   Same requirements as specified for preliminary plan.

5. Design and Traffic Data

   Same requirements as specified for preliminary plans, plus see 6.3.2.1 for guidance regarding sheet border selection.

6. Hydraulic Information, if Applicable

7. Foundations

   Give soil/rock bearing capacity or pile capacity.

   Example for General Plan sheet: Abutments to be supported on HP 10 x 42 steel piling driven to a required driving resistance of 180 tons * per pile as determined by the Modified Gates Dynamic Formula. Estimated 50'-0" long.

   *The factored axial resistance of piles in compression used for design is the required driving resistance multiplied by a resistance factor of 0.5 using modified Gates to determine driven pile capacity.

   Abutments with spread footings to be supported on sound rock with a required factored bearing resistance of "XXX" PSF ***. A geotechnical engineer, with three days notice, will determine the factored bearing resistance by visual inspection prior to construction of the abutment footing.

   *** The factored bearing resistance is the value used for design.

   Repeat the note above on each substructure sheet, except the asterisk (*) and subsequent explanation of factored design resistance need not appear on individual substructure sheets.

   See Table 11.3-5 for typical maximum driving resistance values.

8. Estimated Quantities

   a. Enter bid items and quantities as they appear, and in the order in which they appear in the "Schedule of Bid Items" of the Standard Specifications. Put items not provided for at the bottom of the list. Enter quantities for each part of the structure, (superstructure, each abutment, each pier) under a separate column with a grand total.

      Quantities are to be bid under items for the Structure Type and not by the "B" or "C" numbers. For example, concrete for a multi-cell box culvert exceeding a total length of 20 feet is to be bid under item Concrete Masonry Culverts. As another example, a bridge having a length less than 20 feet would be given a "C" number; however, the concrete bid item is Concrete Masonry Bridges.

   b. For incidental items to be furnished for which there is no bid item, and compensation is not covered by the *Standard Specifications* or *Special Provisions*, note on the plans the most closely related bid item that is to include the cost in the price bid per unit of item. As an example, the cost of concrete inserts is to be included in the price bid per cubic yard of concrete masonry.

9. General Notes

   A standard list of notes is given in 6.3.2.1.1 and 6.3.2.1.2. Use the notes that apply to the structure drawn on the plans.

10. List of Drawings

    Each sheet is numbered sequentially beginning with 1 for the first sheet. Give the sheet number and title of sheet.

11. Bench Marks

    Give the location, description and elevation of the nearest bench mark.

12. Title Block

    Fill in all data for the Title Block except the signature. The title of this sheet is "General Plan". Use the line below the structure number to describe the type of crossing. (Example: STH 15 SB OVER FOX RIVER). See 6.3.2.1 for guidance regarding sheet border selection.

13. Professional Seal

    All final bridge plans prepared by Consultants or Governmental Agencies shall be professionally sealed, signed, and dated on the general plan sheet. If the list of drawings is not on the general plan sheet, the sheet which has the list of drawings shall also be professionally sealed, signed, and dated. This is not required for WisDOT prepared plans, as they are covered elsewhere.

6.3.2.1.1 Plan Notes for New Bridge Construction

1. Drawings shall not be scaled. Bar Steel Reinforcement shall be embedded 2” clear unless otherwise shown or noted.
2. All field connections shall be made with 3/4” diameter friction type high-tensile strength bolts unless shown or noted otherwise.
3. Slab falsework shall be supported on piles or the substructure unless an alternate method is approved by the Engineer.
4. The first or first two digits of the bar mark signifies the bar size.
5. The slope of the fill in front of the abutments shall be covered with heavy riprap and geotextile fabric Type ‘HR’ to the extent shown on sheet 1 and in the abutment details.
6. The slope of the fill in front of the abutments shall be covered with slope paving material to the extent shown on sheet 1 and in the abutment details.
7. The stream bed in front of the abutment shall be covered with riprap as shown on this sheet and in the abutment details.
8. The existing stream bed shall be used as the upper limits of excavation at the piers.
9. The existing ground line shall be used as the upper limits of excavation at the piers.

10. Within the length of the box all spaces excavated and not occupied by the new structure shall be backfilled with Structure Backfill to the elevation and section existing prior to excavation within the length of the culvert.

11. At the backface of abutment all volume which cannot be placed before abutment construction and is not occupied by the new structure shall be backfilled with structure backfill.

12. Concrete inserts to be furnished by the utility company and placed by the contractor. Cost of placing inserts shall be included in the bid price for concrete masonry.

13. Prestressed Girder Bridges - The haunch concrete quantity is based on the average haunch shown on the Prestressed Girder Details sheet.

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

<u>WisDOT policy item:</u>

The note “Dimensions shown are based on the original structure plans” is acceptable. However, any note stating that the contractor shall field verify dimensions is not allowed.

It is the responsibility of the design engineer to use original structure plans, as-built structure plans, shop drawings, field surveys and structure inspection reports as appropriate when producing rehabilitation structure plans of any type (bridges, retaining walls, box culverts, sign structures, etc.). If uncertainty persists after reviewing available documentation, a field visit may be necessary by the design engineer.

1. Dimensions shown are based on the original structure plans.

2. All concrete removal not covered with a concrete overlay shall be defined by a 1 inch deep saw cut.

3. Utilize existing bar steel reinforcement where shown and extend 24 bar diameters into new work, unless specified otherwise.

4. Concrete expansion bolts and inserts to be furnished and placed by the contractor under the bid price for concrete masonry.

5. At "Curb Repair" expose existing reinforcement a minimum of 1 1/2” clear.

6. Existing floor drains to remain in place. Remove top of deck in drain area as directed by the Field Engineer to allow placing and sloping of 1 1/2” concrete overlay.

7. Expansion joint assembly, including anchor studs and hardware shall be paid for in the lump sum price bid as "Expansion Device B-_____” or “Expansion Device Modular B-______”.

8. Clean and fill existing longitudinal and transverse cracks with penetrating epoxy as directed by the Field Engineer.

9. Variations to the new grade line over 1/4” must be submitted by the Field Engineer to the Structures Design Section for review.

10. The contractor shall supply a new name plate in accordance with Section 502.3.11 of the *Standard Specifications* and the standard detail drawings. Name plate to show original construction year.

6.3.2.2 Subsurface Exploration

This sheet is initiated by the Geotechnical Engineer. The following information is required on the sheet. Bridge details are not drawn by the Geotechnical Engineer.

1. Plan View

   Show a plan layout of structure with survey lines, reference lines, pier and abutment locations and location of borings and probings plotted to scale.

   On box culvert structure plans, show three profile lines of the existing ground elevations (along the centerline and outer walls of the box). Scale the information for these lines from the site contour map that is a part of the structure survey report.

2. Elevation

   Show a centerline profile of existing ground elevation.

   Show only substructure units at proper elevation w/no elevations shown. Also show the pile lengths.

   Show the kind of material, its located depth, and the blow count of the split spoon sampler for each boring. Give the blow count at about 5 foot intervals or where there is a significant change in material.

6.3.2.3 Abutments

Use as many sheets as necessary to show details clearly. Each substructure unit should have its own plan sheet(s). Show all bar steel required using standard notations; solid lines lengthwise and solid dots in cross section. Give dimensions for a skewed abutment to a reference line which passes through the intersection for the longitudinal structural reference line and centerline of bearing of the abutment. Give the dimension, from centerline of bearing to backface of abutment along the longitudinal reference line and the offset distance if on a skew. Show the skew angle.

If there is piling, show a complete footing layout giving piling dimensions tied to the reference line. Number all the piles. Give the type of piling, length and required driving resistance. Show a welded field splice for cast in place concrete or steel H piles.

Bridge seats for steel bearings and laminated elastomeric bearings are level within the limits of the bearing plate. Slope the bearing area utilizing non-laminated elastomeric bearings if the slope of the bottom of girder exceeds 1%. Slope the bridge seat between bearings 1” from front face of backwall to front face of abutment. Give all beam seat elevations.

1. Plan View

    a. Place a keyed construction joint near the center of the abutment if the length of the body wall exceeds 50 feet. Make the keyway as large as feasible and extend the horizontal bar steel through the joint.

    b. Dimension wings in a direction parallel and perpendicular to the wing centerline.

    c. Dimension angle between wing and body if that angle is different from the skew angle of the abutment.

2. Elevation

    a. Give beam seat, wing (front face and wing tip), and footing elevations to the nearest .01 of a foot.

    b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

    Place an optional keyed construction joint in the parapet at the bridge seat elevation if there is a parapet.

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

Use as many sheets as necessary to show all details clearly. Each substructure unit should have its own plan sheet(s).

Give dimensions for a skewed pier to a reference line which passes through the intersection of the longitudinal structural reference line and the pier centerline. Show the skew angle. Dimension the centerline spacing of superstructure girders.

1. Plan View

   Show dimensions, footings, cap steps, beam spacings and skew angle.

2. Elevation

   Show dimensions and elevations. Show lengths of all columns for clarity. Give the elevation of the bottom of footings and beam seats. Refer to abutments for detailing bridge seats. Dimension all bar steel and stirrups.

3. Footing Plan

   Show dimensions for pile spacing, pile numbers and reinforcing steel in footing.

4. Bar Steel Listing and Details

5. Pile Splice Detail (If different from abutment only).

6. Cross Section thru Column and Pier Cap

   Detail anchor bolts between reinforcing bars to provide clearance. Long steel bridges may require more clearance. This allows an erection tolerance for the structural steel so that the bar steel is not pierced by the anchor bolts if the bearing is shifted.

6.3.2.5 Superstructure

Use as many sheets as are necessary to show all details clearly. Standard insert sheets are available to show many standard details. The title, project number, and a few basic dimensions are added to these standard sheets.

6.3.2.5.1 All Structures

1. Show the cross-section of roadway, plan view and related details, elevation of typical girder or girders, details of girders, and other details not shown on standard insert sheets. All drawings are to be fully dimensioned and show such sections and views as needed to detail the superstructure completely.

2. For girder bridges:
   Show the total dead load deflections, including composite dead load (without future wearing surface) acting on the composite section, at tenth points of each span. Distribute the composite dead load evenly to all girders and provide one deflection value for a typical interior girder. Chapter 17 – Superstructure-General illustrates three load cases for exterior girder design with raised sidewalks, cases that provide a conservative envelope to ensure adequate girder capacity. However, the above composite dead load distribution should be used for deflection purposes. For prestressed concrete girders, the dead load deflection reported does not include the weight of the girder. See Chapter 24 – Steel Girder Structures for camber and blocking, top of steel elevation and deflection reporting criteria.

A separate deflection value for interior and exterior girders *may* be provided if the difference, accounting for load transfer between girders, warrants multiple values. A weighted distribution of composite dead load could be used for deflection purposes *only*. For example, an extremely large composite load over the exterior girder could be distributed as 40-30-30 percent to the exterior and first two interior girders respectively. Use good engineering judgment to determine whether to provide separate deflection values for individual girder lines. In general, this is not necessary.

For slab bridges:
Provide camber values at the tenth points of all spans. The camber is based on 3 times the deflection of the slab, only. For multi-span bridges, the deflection calculations are based on a continuous span structure since the falsework supports the bridge until the concrete slab has cured.

Deflection and camber values are to be reported to the nearest 0.1 inch, for all girder and slab superstructures.

3. For girder structures, provide finished grade top of deck elevations for each girder line at the tenth points of all spans. Show the top of deck elevations at the outside edge of deck at tenth points. If staged construction, include tenth point elevations along the construction joint. For slab structures, provide the finished grade elevations at the reference line and/or crown and edge of slab at tenth points.

4. Decks of uniform thickness are used on all girders. Variations in thickness are achieved by haunching the deck over each girder. Haunches are formed off the top of the top flange. See the standards for details. In general the minimum haunch depth along the edge of girder is to be 1 1/4” although 2” is recommended to allow for construction tolerances. Haunch depth is the distance from the bottom of the concrete deck to the top of the top flange.

5. Provide a paving notch at each end of all structures for rigid approach pavements. See standard for details.

6. If the structure contains conduit for a deck lighting system, place the conduit in the concrete parapet. Place expansion devices on conduit which passes through structure expansion joints.

7. Show the bar steel reinforcement in the slab, curb, and sidewalk with the transverse spacing and all bars labeled. Show the direction and amount of roadway crown.

8. On bridges with a median curb and left turn lane, water may be trapped at the curb due to the grade slope and crown slope. If this is the case, make the cross slope flat to minimize the problem. Existing pavers cannot adjust to a variable crown line.

9. On structures with modular joints consider cover plates for the back of parapets when aesthetics are a consideration.

10. Provide a table of tangent offsets for the reference line and edges of deck at 10 foot intervals for curved bridges.

6.3.2.5.2 Steel Structures

1. Show the diaphragm connections on steel girders. Show the spacing of rail posts on the plan view.

2. Show a steel framing plan for all steel girders. Show the spacing of diaphragms.

3. On the elevation view of steel girders show dimension, material required, field and shop splice locations, stiffener spacing, shear connector spacing, and any other information necessary to construct the girder. In additional views show the field splice details and any other detail that is necessary.

4. Show the size and location of all weld types with the proper symbols except for butt welds. Requirements for butt welds are covered by A.W.S. Specifications.

5. See Chapter 24 – Steel Girder Structures for camber and blocking, top of steel elevation and deflection reporting criteria.

6. Existing flange and web sizes should be shown to facilitate the sizing of bolts on Rehabilitation Plans.

6.3.2.5.3 Railing and Parapet Details

Standard drawings are maintained by the Structures Development Section showing railing and parapet details. Add the details and dimensions to these drawings that are unique to the structure being detailed. Compute the length along the slope of grade line rather than the horizontal dimension.

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

Show a complete bill of bar steel reinforcement for each unit of the structure. Place this bill on the sheet to which the bars pertain. If the abutments or piers are similar, only one bar list is needed for each type of unit.

Give each bar or group of bars a different mark if they vary in size, length, or location in a unit. Each bar list is to show the mark, number of bars, length, location and detail for each bar. Give bar lengths to the nearest 1” and segment lengths of bent bars to the nearest 1/2”. Show all bar bends and hooks in detail.

Identify all bars with a letter indicating the unit in which the bar is placed - A for abutment, P for pier, S for superstructure. Where units are multiple, each unit should have a different letter. Next use a one or two digit number to sequentially number the bars in a unit. P1008 indicates bar number 08 is a size number 10 bar located in a Pier.

Use a Bar Series Table where a number of bars the same size and spacing vary in length is a uniform progression. Use only one mark for all these bars and put the average length in the table.

Refer to the Standard drawings in Chapter 9 – Materials for more information on reinforcing bars such as minimum bend diameter, splice lengths, bar supports, etc.

When a bridge is constructed in stages, show the bar quantities for each stage. This helps the contractor with storage and retrieval during construction.

6.3.3.2 Box Culverts

Detail plans for box culverts are to be fully dimensioned and have sectional drawings needed to detail the structure completely. The following items are to be shown when necessary:

1. Plan View
2. Longitudinal section
3. Section thru box
4. Wing elevations
5. Section thru wings
6. Section thru cutoff wall
7. Vertical construction joint
8. Bar steel clearance details
9. Header details
10. North point, Bench mark, Quantities
11. Bill of bars, Bar details
12. General notes, List of drawings, Rip rap layout
13. Inlet nose detail on multiple cell boxes
14. Corner details

Bid items commonly used are excavation, concrete masonry, bar steel, rubberized membrane waterproofing, backfill and rip rap. Filler is a non-bid item. In lieu of showing a contour map, show profile grade lines as described for Subsurface Exploration sheet.

See the standard details for box culverts for the requirements on vertical construction joints, apron and cutoff walls, longitudinal construction joints, and optional construction joints.

Show name plate location on plan view and on wing detail.

6.3.3.3 Miscellaneous Structures

Detail plans for other structures such as retaining walls, pedestrian bridges, and erosion control structures are to be detailed with the same requirements as previously mentioned. Multiple sign structure of the same type and project may be combined into a single set of plans per standard insert sheet provisions, and shall be subject to the same requirements for bridge plans.

6.3.3.4 Standard Drawings

Standard drawings are maintained and furnished by the Structures Development Section. These drawings show the common types of details required on the contract plans.

6.3.3.5 Insert Sheets

These sheets are maintained by the Structures Development Section and are used in the contract plans to show standard details.

6.3.3.6 Change Orders and Maintenance Work

These plans are drawn on full size sheets. A Structure Survey Report should be submitted for all maintenance projects, including painting projects and polymer overlay projects. In addition to the SSR, final structure plans on standard sheet borders with the #8 tab should be submitted to BOS in the same fashion as other rehabilitation plans. Painting plans should include at minimum a plan view with overall width and length dimensions, the number of spans, an indication of the number and type of elements to be painted (girders, trusses, etc.), and an elevation view showing what the structure is crossing. The SSR should give a square foot quantity for patchwork painting. For entire bridges or well defined zones (e.g. Paint all girders 5 feet on each side of expansion joints), the design engineer will be responsible for determining the quantity.

6.3.3.7 Name Plate and Bench Marks

WisDOT has discontinued the statewide practice of furnishing bench mark disks and requiring them to be placed on structures. However, WisDOT Region Offices may continue to provide bench mark disks for the contract to be set. Bench mark disks shall be shown on all bridge and larger culvert plans. Locate the bench mark disks on a horizontal surface flush with the concrete. Bench marks to be located on top of the parapet on the bridge deck, above the first right corner of the abutment traveling in the highway cardinal directions of North or East. For multi-directional bridges, locate the name plate on the roadway side of the first right wing or parapet traveling in the highway cardinal directions of North or East. For one-directional bridges, locate the name plate on the first right wing or parapet in the direction of travel. For type “NY”, “W”, “M” or timber railings, name plate to be located on wing. For parapets, name plate to be located on inside face of parapet.

6.3.4 Checking Plans

Upon completion of the design and drafting of plans for a structure, the final plans are usually checked by one person. Dividing plans checking between two or more Checkers for any one structure leads to errors many times. The plans are checked for compliance with the approved preliminary drawing, design, sufficiency and accuracy of details, dimensions, elevations, and quantities. Generally the information shown on the preliminary plan is to be used on the final plans. Revisions may be made to footing sizes and elevations, pile lengths, dimensions, girder spacing, column shapes, and other details not determined at the preliminary stage. Any major changes from the preliminary plan are to be approved by the Structural Design Engineer and Supervisor.

The Checkers check the final plans against the Engineer's design and sketches to ensure all information is shown correctly. The Engineer prepares all sketches and notations not covered by standard drawings. A good Checker checks what is shown and noted on the plan and also checks to see if any essential details, dimensions, or notation have been omitted. The final plan Bid Items should be checked for conformity with those listed in the WisDOT Standard Specifications for Highway and Structure Construction.

The Checker makes an independent check of the Bill of Bars list to ensure the Plan Preparer has not omitted any bars when determining the quantity of bar steel.

Avoid making minor revisions in details or dimensions that have very little effect on cost, appearance, or adequacy of the completed structure. Check grade and bridge seat elevations and all dimensions to the required tolerances. The Checkers make all corrections, revisions, and notations on a print of the plan and return it to the Plan Preparer. The Plan Preparer back checks all marks made by the Checker before changing. Any disagreements are resolved with the Supervisor.

Common complaints received from field staff are dimension errors, small details crowded on a drawing, lettering is too small, and reinforcing bar length or quantity errors.

After the plans are completed, the items in the project folder are separated into the following groups by the Structures Design Engineer:

6.3.4.1 Items requiring a PDF copy for the Project Records (Group A) – Paper Copies to be Destroyed when Construction is Completed.

1. QC/QA sign-off sheet
2. Design computations and computer runs
3. Quantity computations
4. Bridge Special Provisions and STSP’s (only those STSP’s requiring specific blanks to be filled in or contain project specific information)
5. Final Structure Survey Report form (not including photos, cross-sections, project location maps, etc.)

6. Final Geotechnical Report

7. Final Hydrology and Hydraulic computations and structure sizing report

8. Contour map

6.3.4.2 Additional Items to be Destroyed When Construction is Completed (Group B)

1. Miscellaneous correspondence and transmittal letters

2. Preliminary drawings and computations

3. Prints of soil borings and plan profile sheets

4. Shop steel quantity computations*

5. Design checker computations

6. Layout sheets

7. Elevation runs and bridge geometrics

8. Falsework plans*

9. Miscellaneous Test Report

10. Photographs of bridge rehabs

* These items are added to the packet during construction.

6.3.4.3 Items to be Destroyed when Plans are Completed (Group C)

1. All "void" material

2. All copies except one of preliminary drawings

3. Extra copies of plan and profile sheets

4. Preliminary computer design runs

Note that lists for Group A, B & C are not intended to be all inclusive, but serve as starting points for categorizing design material. Items in Group A & B should be labeled separately. Computation of Quantities

6.4 Computation of Quantities

When the final drafting and plan checking is completed, the person responsible for drafting the plans and plans checker are to prepare individual quantity calculations for the bid items listed on the plans. The following instructions apply to the computation on quantities.

Divide the work into units that are repetitive such as footings, columns, and girders. Label all items with a clear description. Use sketches for clarity. These computations may be examined by others in future years so make them understandable.

One of the most common errors made in quantity computation is computing only half of an item which is symmetrical about a centerline and forgetting to double the result.

Following is a list of commonly used bridge quantities. Be sure to use the appropriate item and avoid using incidental items as this is too confusing for the contractor and project manager. The bid item for Abatement of Asbestos Containing Material should be included on the structure plans. Items such as Incentive Strength Concrete Structures, Construction Staking Structure Layout, etc. should not be included on the structure plans.

A column with the title "Bid Item Number" should be the first column for the "Total Estimated Quantities" table shown in the plans. The numbers in this column will be the numbers associated with the bid items as found in the Standard Specification, STSP, and/or Special Provisions.

6.4.1 Excavation for Structures Bridges (Structure)

This is a lump sum bid item. The limits of excavation are shown in the chapter in the manual which pertains to the structural item, abutments, piers, retaining walls, box culverts, etc. If the excavation is required for the roadway, the work may be covered under Excavation Common.

The limits of excavation made into solid rock are the neat line of the footing.

6.4.2 Granular Materials

Granular materials can be bid in units of tons or cubic yards. Structure plans should use the TON bid item for Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch, unless directed otherwise by the Region. The Region may consider use of the CY bid item when contractor-provided tickets may be problematic or when the TON bid item is not used elsewhere on the contract. Other cases may also warrant the use of the CY bid item.

For Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch materials use a 2.0 conversion factor (tons/cubic yard) for compacted TON bid items or use a 1.20 expansion factor (i.e. add an additional 20%) for CY bid items, unless directed otherwise. Refer to the FDM when preparing computations using other granular materials (breaker run, riprap, etc.).

Granular quantities and units should be coordinated with the roadway designer. For some structures, backfill quantities may be negligible to the roadway, while others may encompass a large portion of the roadway cross section and be present in multiple cross sections. A long

MSE retaining wall would be an example of the latter case and will require coordination with the roadway designer.

Generally, granular material pay limits should be shown on all structure plans. This information should be used to generate the estimated quantities and used to coordinate with roadway cross sections and construction details. See Standard Detail 9.01 – Structure Backfill Limits and Notes - for typical pay limits and plan notes.

Refer to 9.10 for additional information about granular materials.

6.4.3 Concrete Masonry Bridges

Show unit quantities to the nearest cubic yard, as well as the total quantity. In computing quantities no deduction is made for metal reinforcement, floor drains, conduits and chamfers less than 2". Flanges of steel and prestressed girders projecting into the slab are deducted.

Deduct the volume of pile heads into footings and through seals for all piling except steel H sections. Deduct the actual volume displaced for precast concrete and cast-in-place concrete piling.

Consider the concrete parapet railing on abutment wing walls as part of the concrete volume of the abutment.

6.4.4 Prestressed Girder Type I (28-Inch; 36-Inch; 36W-Inch; 45W-Inch; 54W-Inch; 72W-Inch, 82W-Inch)

Record the total length of prestressed girders to the nearest 1 foot.

6.4.5 Bar Steel Reinforcement HS Bridges or Bar Steel Reinforcement HS Coated Bridges

Record this quantity to the nearest 10 lbs. Designate if bar steel is coated. Include the bar steel in C.I.P. concrete piling in bar steel quantities.

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

Record this quantity to the nearest 10 lbs. Bar weight shall be assumed to be equivalent to Bar Steel Reinforcement HS Bridges or Bar Steel Reinforcement HS Coated Bridges bid items.

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

6.4.8 Bearing Pads Elastomeric Non-Laminated or Bearing Pads Elastomeric Laminated or Bearing Assemblies Fixed (Structure) or Bearing Assemblies Expansion (Structure)

Record as separate item with quantity required. Bid as Each.

6.4.9 Piling Test Treated Timber (Structure)

Record this quantity as a lump sum item. Estimate the pile lengths by examining the subsurface exploration sheet and the Site Investigation Report. Give the length and location of test piles in a footnote. Do not use this quantity for steel piling or concrete cast-in-place piling.

6.4.10 Piling CIP Concrete Delivered and Driven ___-Inch, Piling Steel Delivered and Driven ___ -Inch

Record this quantity in feet for Steel and C.I.P. types of piling delivered and driven. Timber piling are Bid as separate items, delivered and driven. Pile lengths are computed to the nearest 5.0 foot for each pile within a given substructure unit, unless a more exact length is known due to well defined shallow rock (approx. 20 ft.), etc.. Typically, all piles within a given substructure unit are shown as the same length.

The length of foundation piling driven includes the length through any seal and embedment into the footing. The quantity delivered is the same as quantity driven. For trestle piling the amount of piling driven is the penetration below ground surface.

Oil field pipe is allowed as an alternate on all plans unless a note is added in the General Notes stating it is not allowed on that specific project.

6.4.11 Preboring CIP Concrete Piling or Steel Piling

Record the type, quantity in feet. Calculate to the nearest lineal foot per pile location.

6.4.12 Railing Steel Type (Structure) or Railing Tubular Type (Structure)

Record the type and quantity, bid in lineal feet. For bridges, the railing length should be horizontal length shown on the plans. For retaining walls, use the length along the top of the wall. Calculate railing lengths as follows:

- Steel Railing Type ‘W’ – CL end post to CL end post
- Tubular Railing Type ‘H’ – CL end plate to CL end plate
- Combination Railing Type ‘3T’ – CL end post to CL end post + (2’-5”) per railing
- Tubular Railing Type ‘M’ – CL end post to CL end post + (4’-6”) per railing
- Combination Railing Type ‘Type C1-C6’ – CL end rail base plate to CL end rail base plate
- Tubular Steel Railing Type NY3&4 – CL end post to CL end post + (4’-10”) per railing

6.4.13 Slope Paving Concrete or Slope Paving Crushed Aggregate or Slope Paving Select Crushed Material

Record this quantity to the nearest square yard. Deduct the area occupied by columns or other elements of substructure units.

6.4.14 Riprap Medium, Riprap Heavy or Grouted Riprap, Riprap Light

Record this quantity to the nearest 5 cubic yards.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

6.4.16 Floordrains Type GC, Floordrains Type H, or Floordrains Type WF

Record the type and number of drains. Bid as Each.

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

For “Expansion Device” and “Expansion Device Modular”, bid the items in lineal feet. The distance measured is from flowline to flowline along the skew (do not include turn-ups into parapets or medians).

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

6.4.21 Conduit Rigid Metallic __-Inch or Conduit Rigid Nonmetallic Schedule 40 -Inch

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

Record these quantities to the nearest square yard. Preparation Decks Type 1 should be provided by the Region. Estimate Preparation Decks Type 2 as 40% of Preparation Decks Type 1. Deck preparation areas shall be filled using Concrete Masonry Overlay Decks, Concrete Masonry Deck Repair, or with an appropriate deck patch. See Chapter 40 Standards.

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

Record this quantity to the nearest cubic yard. Estimate the quantity by using a thickness measured from the existing ground concrete surface to the plan gradeline. Calculate the minimum overlay thickness and add ½" for variations in the deck surface. Provide this average thickness on the plan, as well. Usually 1" of deck surface is removed by grinding. Include deck repair quantities for Preparation Decks Type 1 & 2 and Full-Depth Deck Repair. Use 2-inch thickness for each Preparation area and ½ the deck thickness for Full-Depth Deck Repairs in areas of deck preparation (full-depth minus grinding if no deck preparation).

6.4.28 Removing Old Structure STA. XX + XX.XX

Covers the entire or partial removal of an existing structure. Bid as Lump Sum.

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

Attachment assembly for Beam Guard at the termination of concrete parapets. Bid as each.

6.4.30 Steel Diaphragms (Structure)

In span diaphragms used on bridges with prestressed girders. Bid as each.

6.4.31 Welded Stud Shear Connectors X -Inch

Total number of shear connectors with the given diameter. Bid as each.

6.4.32 Concrete Masonry Seal

Seal concrete bid to the nearest cubic yard. Whenever a concrete seal is shown on the plans, then "Cofferdams (Structure)" is also to be a bid item.

6.4.33 Geotextile Fabric Type

List type of fabric. Type HR is used in conjunction with Heavy Riprap. Bid in square yards.

6.4.34 Concrete Adhesive Anchors

Used when anchoring reinforcing bars into concrete. Bid as each.

6.4.35 Piling Steel Sheet Permanent Delivered or Piling Steel Sheet Permanent Driven

Record this quantity to the nearest square foot for the area of wall below cutoff.

6.4.36 Piling Steel Sheet Temporary

This quantity is used when the designer determines that retention of earth is necessary during excavation and soil forces require the design of steel sheet piling. This item is seldom used now that railroad excavations have a unique SPV.

Record this quantity to the nearest square foot for the area from the sheet pile tip elevation to one foot above the retained grade.

6.4.37 Temporary Shoring

This quantity is used when earth retention may be required and the method chosen is the contractor's option.

Measured as square foot from the ground line in front of the shoring to a maximum of one foot above the retained grade. For the estimated quantity use the retained area (from the ground line in front of the shoring to the ground line behind the shoring, neglecting the additional height allowed for measurement).

6.4.38 Concrete Masonry Deck Repair

Record this quantity to the nearest cubic yard. Use 2-inch thickness for each Preparation area and ½ the deck thickness for Full-Depth Deck Repairs.

6.4.39 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per SY of Preparation Decks Type 1.

6.4.40 Removing Bearings

Used to remove existing bearings for replacement with new expansion or fixed bearing assemblies. Bid as each.

6.4.41 Ice Hot Weather Concreting

Used to provide a mechanism for payment of ice during hot weather concreting operations. See FDM 19-7-1.2 for bid item usage guidance and quantity calculation guidance. Bid as LB and round to the nearest 5 lbs.

6.5 Production of Structure Plans by Consultants, Regional Offices and Other Agencies

The need for structures is determined during the Preliminary Site Survey and recorded in the Concept Definition or Work Study Report. On Federal (FHWA) or State Aid Projects (including maintenance projects), a completed Structure Survey Reports, preliminary and final plans are submitted to the Bureau of Structures with a copy forwarded to the Regional Office for review and approval prior to construction. Structure and project numbers are provided by the Regional Offices. In preparation of the structural plans, the appropriate specifications and details recommended by the Bureau of Structures are to be used. If the consultant elects to modify or use details other than recommended, approval is required prior to their incorporation into the final plans.

On all Federal or State Aid Projects involving Maintenance work, the Concept Definition or Work Study Report, the preliminary and final bridge reconstruction plans shall be submitted to the Bureau of Structures for review.

Consultants desiring eligibility to perform engineering and related services on WisDOT administered structure projects must have on file with the Bureau of Structures, an electronic copy of their current Quality Assurance/Quality Control (QA/QC) plan and procedures. The QA/QC plan and procedures shall include as a minimum:

- Procedures to detect and correct bridge design errors before the design plans are made final.
- A means for verifying that the appropriate design calculations have been performed, that the calculations are accurate, and that the capacity of the load-carrying members is adequate with regard to the expected loads on the structure.
- A means for verifying the completeness, constructability and accuracy of the structure plans.
- Verification that independent checks, reviews and ratings were performed.

A QA/QC verification summary sheet is required as part of every final structure plan submittal, demonstrating that the QA/QC plan and procedures were followed for that structure. The QA/QC verification summary sheet shall include the signoff or initialing by each individual that performed the tasks (design, checking, plan review, technical review, etc.) documented in the QA/QC plan and procedures. The summary sheet must be submitted with the final structure plans as part of the e-submit process.

6.5.1 Approvals, Distribution, and Work Flow

Consultant	Meet with Regional Office and/or local units of government to determine need.
	Prepare Structure Survey Report including recommendation of structure type.
Geotechnical Consultant	Make site investigation and prepare Subsurface Investigation Report.

Consultant	Submit hydrology report via Esubmit or as an email attachment to the supervisor of the Consultant Review and Hydraulics Unit. Submit 60 days prior to preliminary plan submittal.
	Prepare preliminary plans according to 6.2.
	Coordinate with Region and other agencies per 6.2.3.
	Submit preliminary plans, SSR and supporting documents via e-submit for review and approval of type, size and location.
Structures Design Section	Record project information in HSIS.
	Review hydraulics for Stream Crossings.
	Review Preliminary Plan. A minimum of 30 days to review preliminary plans should be expected.
	Coordinate with other agencies per 6.2.3.
	Return preliminary plans and comments from Structures Design Section and other appropriate agencies to Consultant with a copy to the Regional Office.
	Forward Preliminary Plan and Hydraulic Data to DNR.
Consultant	Modify preliminary plan as required, and provide explanation for preliminary comments not incorporated in final plan.
	Prepare and complete final design and plans for the specified structure.
	Write special provisions.
	At least **two months** in advance of the PS&E date, submit the required final design documents via e-submit per 6.5.3.
Structures Design Section	Determine which final plans will be reviewed and perform quality assurance review as applicable.
	For final plans that are reviewed, return comments to Consultant and send copy to Regional Office, including FHWA as appropriate.
Consultant	Modify final plans and specifications as required.
	Submit modified final plans via e-submit as required.
Structures Design Section	Review modified final plans as applicable.
	Sign final plans and send performance evaluation form to Region and Consultant.
Bureau of Project Development	Prepare final accepted structure plans for pre-development contract administration.

Consultant	If a plan change is needed after being advertised but before being let, an addendum is required per FDM 19-22-1 and 19.1 Attachment 1.2.
Structures Design Section	Review structure addendum as applicable.
	Sign structure addendum.
Bureau of Project Development	Distributes structure addendum to bidders.
Consultant	If a plan change is required after being let, a post-let revision is required per 6.5.5.
Structures Design Section	Review post-let revision as applicable.
	Stamp post-let revision plan as accepted.
	Delivers revised plan to DOT construction team for distribution.

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Preliminary Plan Requirements

The Consultant prepares the Structure Survey Report for the improvement. Three types of Structure Survey Reports are available at the Regional Offices and listed in 6.2.1 of this Chapter. Preliminary layout requirements are given in 6.2.2. The Preliminary Plan exhibits are as follows:

1. Hydrology Report
2. Structure Survey Report
3. Preliminary plan, including log borings shown on the subsurface exploration sheet
4. Evaluation of subsurface investigation report
5. Contour map
6. Plan and profile, and typical section for roadway approaches
7. Hydraulic/Sizing Report (see Chapter 8 - Hydraulics) and hydraulic files are required for stream crossing structures
8. County map showing location of new and/or existing structures and FEMA map
9. Any other information or drawings which may influence location, layout or design of structure, including DNR initial review letter and photographs

6.5.3 Final Plan Requirements

The guidelines and requirements for Final Plan preparation are given in 6.3. The Load Rating Summary form and On-Time Submittal form can be found on the Bureau of Structures' Design and Construction webpage. The following files are included as part of the final-plan submittal:

1. Final Drawings
2. Design and Quantity Computations

   For all structures for which a finite element model was developed, include the model computer input file(s).

3. Special Provisions covering unique items not in the Standard Specifications or Standardized Special Provisions (STSP).
4. QA/QC Verification Sheet
5. Inventory Data Sheet
6. Bridge Load Rating Summary Form
7. LRFD Input File (Excel ratings spreadsheet)
8. On-Time Improvement Form

   The On-Time Improvement form is required to be submitted if either of the following situations occur:

   - If the first version of a final structure plan is submitted after the deadline of two months prior to the PSE date.
   - If any version of a final structure plan is re-submitted after the deadline of two months prior to the PSE date. However this form is not required when the re-submit is prompted by comments from the Consultant Review Unit. The form is also not necessary when submitting addenda or post-let revisions.

6.5.4 Addenda

Addenda are plan and special provision changes that occur after the bid package has been advertised to potential bidders. See FDM 19-22-1 for instruction on the addenda process.

6.5.5 Post-Let Revisions

Post-let revisions are changes to plan details after the contract is awarded to a bidder. ESubmit only the changed plan sheets, not the entire plan set. The changes to the plan sheet shall be in red font, and outlined by red clouding. The revision box shall also be filled in with red font. Each sheet shall be 11x17, PE stamped, signed, and dated on the date of submittal.

6.5.6 Local-Let Projects

Local-let projects that are receiving State or Federal funding shall be submitted to and reviewed by the Consultant Review Unit in the same way as a State-let project. Final structure plans accepted and signed by the Consultant Review Unit will be returned to the Designer of Record and to the Region for incorporation into the local contract package.

6.6 Structures Data Management and Resources

6.6.1 Structures Data Management

The following items are part of the Data Management System for Structures. The location is shown for all items that need to be completed in order to properly manage the Structure data either by Structures Design personnel for in-house projects or consultants for their designs.

1. Structure Survey Report - Report is submitted by Region or Consultant and placed in the individual structure folder in HSI by BOS support staff.

2. Subsurface Exploration Report - Report is submitted by WisDOT Geotechnical Engineering Unit and placed in the individual structure folder in HSI by BOS support staff.

3. Hydraulic and Scour Computations, Contour Maps and Site Report - Data is assembled by the BOS Consultant Review & Hydraulics Unit and placed in the individual structure folder in HSI by BOS support staff.

4. Designer Computations and Inventory Superstructure Design Run (Substructure computer runs as determined by the Engineer). The designers record design, inventory, operating ratings and maximum vehicle weights on the plans.

5. Load Rating Input File and Load Rating Summary sheet - The designer submits an electronic copy of the input data for load rating the structure to the Structures Development Section. (For internal use, it is located at //H32751/rating.)

6. Structure Inventory Form (Available under “Inventory & Rating Forms” on the HSI page of the BOS website). New structure or rehabilitation structure data for this form is completed by the Structural Design Engineer. It is E-submitted to the Structures Development Section for entry into the File.

7. Pile Driving Reports - An electronic copy of Forms DT1924 (Pile Driving Data) and DT1315 (Piling Record) are to be submitted to the Bureau of Structures by email to “DOTDTSDStructuresPiling@dot.wi.gov ”. These two documents will be placed in HSI for each structure and can be found in the “Shop” folder.

8. Final Shop Drawings for steel bridges (highway and pedestrian), sign bridges, floor drains, railings, all steel joints, all bearings, high-mast poles, prestressed girders, prestressed boxes, noise walls and retaining walls. Metals Fabrication & Inspection Unit or others submit via email to the Structures Development Section at “DOTDLDTSDSTRUCTURESRECORDS@DOT.WI.GOV ”. This process does not, however, supersede submission processes in place for specific projects.

9. Mill Tests, Heat Numbers and Shop Inspection Reports for all Steel Main Members. Metals Fabrication & Inspection Unit electronically files data into HSI

10. As-Let Plans: After bid letting, a digital image of the As-Let plans are placed in a computer folder in the Bureau of Project Development (BPD). BOS office support staff

extract a digital copy of the As-Let structure plans and place it in the structure folder for viewing on HSI.

11. As-Built Plans: As-Built structure plans shall be prepared for all let structure projects, new or rehabilitation. The structures with prefix ‘B’, ‘P’, ‘C’, ‘M’, ‘N’, ‘R’ and ‘S’ shall have As-Built plans produced after construction. The As-Built shall be prepared in accordance with Section 1.65.14 of the Construction and Materials Manual (CMM). These plans are located on a network drive and be viewed in DOTView GIS. BOS staff will ensure that the proper BOS folder (\\dotstrc\04bridge) has a copy of these plans for viewing in HSI.

12. Inspection Reports - A certified bridge inspector enters the initial and subsequent inspection data into HSI.

Initial	Underwater (UW-Probe/Visual)
Routine Visual	Movable
Fracture Critical	Damage
In-Depth	Interim
Underwater (UW)-Dive	Posted
Underwater (UW)-Survey	Structure Inventory and Appraisal

Table 6.6-1

Various Inspection Reports

** HSI – Highway Structures Information System – The electronic file where bridge data is stored for future use.

6.6.2 Resources

The following items are available for assistance in the preparation of structure plans on the department internet sites:

http://on.dot.wi.gov/dtid_bos/extranet/structures/LRFD/index.htm

Bridge Manual
Highway Structures Information System (HSI)
Insert sheets
Standard details
Posted bridge map
Standard bridge CADD files
Structure survey reports and check lists
Structure costs
Structure Special Provisions

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/manuals.aspx

Facilities Development Manual
Standard Specifications for Highway and Structures Construction
Construction and Materials Manual

Additional information is available on the AASHTO and AREMA websites listed below:

http://bridges.transportation.org

https://www.arema.org/

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Table of Contents

7.1 Introduction

Disclaimer:
This chapter is in the early stages of development. The information is limited and will develop over time. The intent of this chapter is to provide guidance to designers, but is far from all-inclusive.

The purpose of the Accelerated Bridge Construction (ABC) Chapter is to provide guidance for the planning and implementation of projects that may benefit from the application of rapid bridge construction technologies and methods. This chapter was prepared to provide planners and engineers with a basic understanding of different ABC methods available, help guide project specific selection of ABC methods, and to encourage the use of the ABC methods described in this chapter.

7.1.1 WisDOT ABC Initiative

The Department's mission is to provide leadership in the development and operation of a safe and efficient transportation system. One of our values relates to Improvement - Finding innovative and visionary ways to provide better products and services and measure our success. The application of Accelerated Bridge Construction (ABC) is consistent with our Mission and Values in promoting efficient development and operation of the transportation system through innovative bridge construction techniques that better serve the public. This service may manifest as safer projects with shorter and less disruptive impacts to the traveling public, and potential cost savings.

WisDOT is following the Federal Highway Administration's (FHWA) Every Day Counts initiative "aimed at shortening project delivery, enhancing the safety of our roadways, and protecting the environment." Two of the five major methods that the FHWA has emphasized as accelerating technologies are Prefabricated Bridge Elements and Systems (PBES) and Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS). These accelerating technologies are incorporated in the following sections in this chapter, namely: Prefabricated Bridge Elements, Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS), Self Propelled Modular Transporters (SPMTs) and Lateral Sliding (both SPMTs and Lateral Sliding are classified as Prefabricated Bridge Systems). WisDOT has had success using GRS-IBS and Prefabricated Bridge Elements, and is always looking for new technologies to improve construction and reduce impacts to traffic. For more information on the Every Day Counts Initiative, refer to www.fhwa.dot.gov/everydaycounts.

7.1.2 ABC Overview

In essence, ABC uses different methods of project delivery and construction to reduce the project schedule, on-site construction time, and public impact. With the ever increasing demand on transportation infrastructure, and the number of bridges that are approaching the end of their service lives, the need for ABC becomes more apparent.

Three main benefits of using ABC methods include minimized impact to traffic, increased safety during construction, and minimized impacts in environmentally sensitive areas. Where conventional bridge construction takes months or years, a bridge utilizing ABC may be placed in a matter of weeks, days, or even a few hours depending on the methods used. ABC methods are generally safer than conventional construction methods because much of the construction can be done offsite, away from traffic. Quality can also be improved because the construction is often completed in a more controlled environment compared to on-site conditions. On the other hand, as with the implementation of all new technologies, the use of ABC comes with challenges that need to be overcome on a project-specific basis.

Oftentimes accelerating the schedule increases the cost of the project. This increased project delivery cost can be offset by reductions in road user costs. In some states, it has been shown that a high percentage of the public approves the use of ABC knowing that the cost can be significantly higher.

WisDOT policy item:

Prior to the implementation of ABC methods on a project, contact the Bureau of Structures Development Section Chief for discussion, resources, and approval.

7.1.3 Accelerated Bridge Construction Technology

Acronym/Term	Definition
ABC (Accelerated Bridge Construction)	Bridge construction methods that use innovative planning, design, materials, and construction techniques in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges.
AC (Alternative Contracting)	Nontraditional project delivery systems, bidding practices, and specifications that may be used to reduce life-cycle costs, improve quality, and accelerate the delivery of construction projects.
BSA (Bridge Staging Area)	Location where a bridge is constructed near the final location for the bridge, where the traveling public is not affected. The bridge can be moved from the staging area to the final location with SPMTs or by sliding.
CM/GC (Construction Manager/General Contractor)	Hybrid of the DBB and D/B processes that allows the owner to remain active in the design process, while the risk is still taken by the general contractor. This method is not an option for WisDOT administered projects.
D/B (Design/Build)	Accelerated project delivery method where one entity (the “designer-builder”) assumes responsibility for both the design and construction of a project. This method is not an option for WisDOT administered projects.

DBB (Design-Bid-Build)	Traditional project delivery method where the owner contracts out the design and construction of a project to two different entities.
EDC (Every Day Counts)	Initiative put forth by FHWA designed to identify and deploy innovation aimed at shortening project delivery, enhancing the safety of our roadways, and protecting the environment.
GRS-IBS (Geosynthetic Reinforced Soil – Integrated Bridge System)	An ABC technology that uses alternating layers of compacted granular fill material and fabric sheets of geotextile reinforcement to provide support for the bridge in place of a traditional abutment.
LBDB (Low Bid Design Build)	A type of D/B where the design and construction service is bundled into a single contract awarded to the lowest competent and responsible bidder.
PBES (Prefabricated Bridge Elements and Systems)	Structural components of a bridge or bridge system that are constructed offsite, or near-site of a bridge that reduce the onsite construction time and impact to the traveling public relative to conventional construction methods.
Pick Points	Locations where the SPMTs will lift and carry the bridge.
Program Initiative	The use of ABC methods to facilitate research, investigate technology, develop familiarity, or address other stakeholder needs.
Road User Costs	Costs pertaining to a project alternative borne by motorists and the community at-large as a result of work zone activity. (FDM 11-50-32)
SPMTs (Self Propelled Modular Transporters)	Remote-controlled, multi-axle platform vehicles capable of transporting several thousand tons of weight.
Stroke	Distance an SPMT can raise or lower its platform.
TMP (Transportation Management Plan)	A set of coordinated transportation management strategies that describes how they will be used to manage work zone impacts of a road project. (FDM 11-50-5)
TP (Travel Path)	Course that the SPMTs travel to carry the completed structure from the staging area to the final location.

Table 7.1-1
ABC Terminology

7.1.4 ABC Methods

7.1.4.1 Prefabricated Bridge Elements

Prefabricated bridge elements are a commonly used ABC method and can be incorporated into most bridge projects as a form of accelerated construction. Concrete bridge elements are prefabricated, transported to the construction site, placed in the final location, and tied into the structure. An entire bridge can be composed of prefabricated elements, or single bridge elements can be prefabricated as the need arises. Prefabricated bridge elements can also be used in combination with other accelerated bridge construction methods. Commonly used prefabricated bridge elements are prestressed concrete girders (including I-girders, adjacent inverted T-beams, and boxes), full depth and partial depth deck panels, abutments, pier caps, pier columns, and footings, as well as precast three-sided and four-sided box culverts.

For all prefabricated bridge elements, shop drawings shall be submitted by email to the Bureau of Structures Development Section Chief.

<u>Figure 7.1-1</u>
Prefabricated Pier Cap

Figure 7.1-2
Prefabricated Abutment

Prefabricated bridge elements are used to mitigate the on-site time required for concrete forming, rebar tying and concrete curing, saving weeks to months of construction time. Deck beam elements eliminate conventional onsite deck forming activities. To reduce onsite deck forming operations, deck beam elements are typically placed in an abutting manner. Prefabricated elements are often of higher quality than conventional field-constructed elements, because the concrete is cast and cured in a controlled environment. The elements are often connected using high strength grout, and post-tensioning or pretensioning. Because some previous prefabricated bridge element connections have had problems, close attention should be given to these connections.

7.1.4.1.1 Precast Piers

Precast concrete piers are optimally used when constructed adjacent to traffic. This application can be best visualized for a two span bridge with a pier located between median barriers. The use of precast piers minimizes traffic disruptions and construction work near traffic.

7.1.4.1.2 Application

Precast concrete piers have successfully been used on past projects. However, these projects did not allow the use of cast-in-place concrete piers which is currently not practical for most projects. An approach that allows for either cast-in-place or precast construction (or a combination thereof) after the contract has been awarded provides contractors greater flexibility to meet schedule demands, provides a safer work environment, and has the potential to reduce costs.

Optional precast concrete pier elements are currently being used on the I-39/90 Project. To aid in the continued development of precast piers, several bridges on the I-39/90 Project required the use of precast pier elements. These mandatory locations will follow the optional precast pier requirements, but prohibit cast-in-place construction. The remaining I-39/90 Project bridges, unless provided an exception, are being delivered as traditional cast-in-place piers with a noted allowance for the contractor to select a precast option. The precast option provides the Project Team and contractors with more flexibility while requiring minimal coordination with designers and the Bureau of Structures.

WisDOT policy item:

At this time, evaluation and plan preparations for accommodating a noted allowance for a precast pier option as indicated in this section is only required for I-39/90 Project bridges. All other locations statewide may consider providing a noted allowance for a precast option. Contact the Bureau of Structures Development Section for further guidance.

In some cases precast piers may not be suitable for a particular bridge location and there are specific limitations that can cause concern. The designer shall investigate the potential viability of precast pier elements for any proposed bridge. The designer should be aware of the common criteria for use and the limitations of the pier system. Some specific limitations for the optional precast pier element usage are the following:

- Piers shall be designed to allow either cast-in-place or precast concrete construction, but with only cast-in-place detailed on the plans. Differences between construction methods shall be limited to pier column connections, beam seats details, and diaphragm details. If the pier configuration is not able to reasonably accommodate interchangeability between the two constructions optional piers may be exempt from the precast option.
- Multi-column piers (3x4 ft rectangular) grade separations over roadways only.
- Fixed piers supporting prestressed concrete girders only.
- Precast elements shall be limited to 90 kips.
- Deep foundations are recommended when multiple pier caps are used. Shallow foundations may be considered if differential settlement is not expected.
- Integral barriers or crashwalls are currently excluded from the precast option.
- Applications where the top of the footing may become submerged are prohibited.

An exception to the precast pier option may be given by the Bureau of Structures.

7.1.4.1.3 Design Considerations

Precast concrete piers shall be designed in conformance with the current *AASHTO LRFD*, in accordance with the WisDOT Bridge Manual, and as given in the Special Provisions.

The optional precast pier allowance shall be established as prescribed in the optional precast pier details and specifications to envelope design requirements between precast and cast-in-place concrete construction. Contract plans shall follow a traditional cast-in-place delivery, with the exception of a noted allowance for precast piers. If the contractor selects the precast option, the contractor shall submit shop drawings, sealed by a professional engineer, to the Bureau of Structures. The fabrication shall be in conformance with the current *AASHTO LRFD*, in accordance to the Bridge Manual, and as given in the Special Provisions. Payment for the precast option will be paid using the cast-in-place concrete bid items.

Refer to Chapter 7 Standards for additional design considerations.

7.1.4.2 Geosynthetic Reinforced Soil – Integrated Bridge Systems (GRS-IBS)

Geosynthetic Reinforced Soil-Integrated Bridge Systems (GRS-IBS) are composed of two main components: Geosynthetic Reinforced Soil (GRS) and Integrated Bridge Systems (IBS). GRS is an engineered fill of closely spaced alternating layers of compacted fill and geosynthetic reinforcement that eliminates the need for traditional concrete abutments. IBS is a quickly-built, potentially cost-effective method of bridge support that blends the roadway into the superstructure using GRS technology. This integration system creates a transition area that allows for uniform settlement between the bridge substructure and the roadway approach, alleviating the “bump at the bridge” problem caused from uneven settlement. The result of this system is a smoother bridge approach.

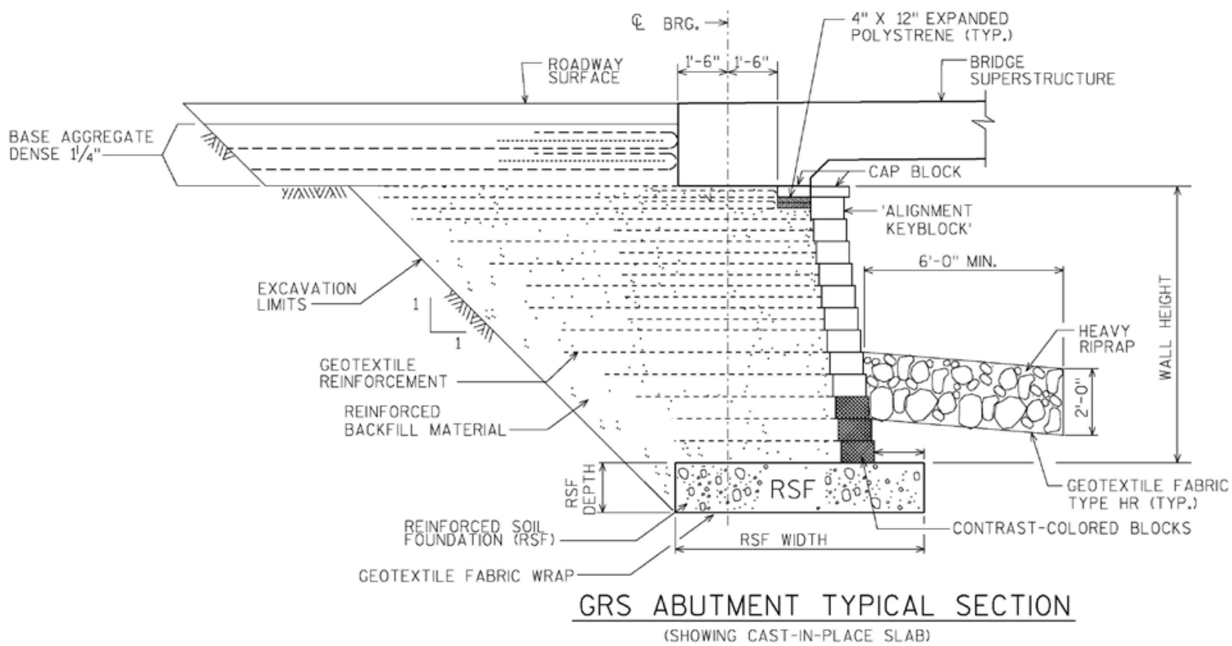

Figure 7.1-3
GRS-IBS Typical Cross Section

Figure 7.1-4
GRS-IBS Structure

Figure 7.1-5
GRS Abutment Layer During Construction

FHWA initially developed this accelerated construction technology, and the first bridge constructed in Wisconsin using the GRS-IBS technology was built in the spring of 2012. This structure (including structure numbers B-9-380, R-9-13, and R-9-14) is located on State Highway 40 in Chippewa County. This structure utilized a single-span cast-in-place concrete slab, which is the first of its kind in the nation. This structure was closely monitored for two years to assess its performance.

This technology has several advantages over traditional bridge construction methods. A summary of the benefits of using GRS-IBS technology include the following:

1. Reduced Construction Time: Due to the simplicity of the design, low number of components, and only requiring common construction equipment to construct, the abutments can be rapidly built.

2. Potential Reduced Construction Costs: Compared to typical bridge construction in Wisconsin, GRS-IBS abutments can achieve significant cost savings. Nationwide, the potential cost savings is reported to be between 25 to 60% over traditional methods. The savings comes largely from the reduced number of construction steps, readily available and economical materials, and the need of only basic tools and equipment for construction.

3. Lower Weather Dependency: GRS-IBS abutments utilize only precast modular concrete facing blocks, open-graded backfill, and geotextile reinforcement in the basic design. The abutments can be constructed in poor weather conditions, unlike cast-in-place concrete, reducing construction delays.

4. Flexible Design: The abutment designs are simplistic and can be easily field-modified where needed to accommodate a variety of field conditions.

5. Potential Reduced Maintenance Cost: Since there are fewer parts to GRS-IBS abutments, overall maintenance is reduced. In addition, when repairs are needed, the materials are typically readily available and the work can be completed by maintenance staff or a variety of contractors.

6. Simpler Construction: The basic nature of the design demands less specialized construction equipment and the materials are usually readily available. Contractor capability and capacity demands are also reduced, allowing smaller and more diverse contractors to bid and complete the work.

7. Less Dependent on Quality Control: GRS-IBS systems are simple and basic in both their design and construction. Lack of technically challenging components and construction methods results in higher overall quality, reducing the probability of quality control related problems.

8. Minimized Differential Settlement: The GRS-IBS system is designed to integrate the structure with the approach pavement. Even though settlements can accumulate, differential settlement between the superstructure and the transition pavement is small. This can substantially reduce the common “bump at the bridge” that can be felt when traveling over traditional bridge transitions.

For more information, see Section 7.3, WisDOT Standard Details 7.01 and 7.02, and the Department's specification.

7.1.4.2.1 Design Standards

GRS Abutments shall be designed in conformance with the current *AASHTO Load and Resistance Factor Design Specifications* (AASHTO LRFD) and in accordance with the WisDOT Bridge Manual and the *FHWA Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide*.

7.1.4.2.2 Application

In some cases GRS-IBS abutments may not be suitable for a particular bridge location and there are specific limitations that can cause concern. As with any preliminary bridge planning, the site should be thoroughly investigated for adequacy. The designer shall investigate the potential viability of using of GRS-IBS for any proposed bridge. The designer should be aware of the common criteria for use and the limitations of GRS-IBS systems. Some of the common criteria for usage of GRS-IBS are the following:

1. Scour potential at the abutment locations has been evaluated and is within acceptable limits
2. Water velocities are less than 5 ft/s
3. Adequate freeboard is provided (See Bridge Manual Chapter 8.3.1.5)
4. Soil conditions permit shallow foundations.
5. Low-volume roadways
6. Single span structure with a span length less than 90 feet
7. Abutment wall height less than 22 feet (measured at the maximum wall height, from the top of the RSF to the top of the wall)
8. Wingwalls are parallel to roadway
9. Maximum skew angle of 15°
10. Short and long term settlements are tolerable
11. Differential settlement along the length of the abutment is tolerable to avoid twisting of the superstructure
12. Suitable construction materials available

7.1.4.2.3 Design Considerations

7.1.4.2.3.1 Hydraulics

Similar to any bridge spanning a waterway, the hydraulic conditions must be evaluated. The integrity of this system is very susceptible to scouring and undercutting of the Reinforced Soil Foundation (RSF) which could lead to further erosion and movement of the backfill in the GRS mass, causing settlement and possible structural failure.

WisDOT policy item:

The use of GRS-IBS is subject to prior-approval by the Bureau of Structures for hydraulic design. Evaluation of scour vulnerability will include assessment of long-term aggradation and degradation, potential for lateral migration of the stream, and calculation of contraction scour and abutment scour. The conservative nature of abutment scour calculations is acknowledged. Placement of adequately designed permanent scour countermeasures will be required to resist calculated scour.

In some cases of bridge replacement, the new GRS-IBS abutments can be constructed behind old abutments which can be left partially in place to promote scour protection for the RSF and GRS mass. Rip-rap, gabion mattresses and other traditional permanent counter measures can also be used.

To help bridge inspectors with scour detection, the lower rows of facing block below proposed grade should have an accent color (typically red, either integral or stained color treatment) that will become visible if scour is occurring. The accented colors provide a visual cue to inspectors that movement of soils has occurred. The top of the contrast-colored blocks shall be placed 2-3 block courses below the top of riprap elevation.

7.1.4.2.3.2 Reinforced Soil Foundation (RSF) and Reinforced Soil Mass

In the GRS-IBS system, bridge seat loads (including dead loads, live loads, etc.) and the weight of the GRS mass and facing blocks comprise the vertical loads that are carried by the RSF and ultimately transmitted to the soil. The vertical bridge seat loads are transferred to the RSF via the GRS mass. The facing blocks only carry their self-weight. Horizontal earth pressure forces are resisted by the GRS mass and little horizontal forces are carried by the facing blocks.

As with any bridge design, proper subsurface exploration should be conducted to ascertain the soil types and layer thicknesses in the vicinity of the proposed site. Laboratory testing may also be necessary to help determine the soil properties and provide the magnitude and time rate of total and differential settlements that may occur.

The external stability of the RSF and reinforced soil mass should be checked for failure against sliding, bearing capacity, and global stability. Due to the behavior of the reinforcement within the soil mass, overturning is an unlikely failure mode, but needs to be checked. The internal stability of the GRS mass should also be checked for bearing capacity, deformations, and the required reinforcement strength. FHWA (1) has provided general guidelines for GRS-IBS ultimate bearing capacities and the predicted deformations when using the prescribed material properties (geotextile, backfill, etc.) and geometry (layer spacings, wall height, etc.). In addition, anticipated settlements should be included when designing for vertical clearance. Under the conditions recommended by FHWA (1), creep in the geotextile reinforcement is typically negligible since the sustained stresses are redistributed and relatively low and reduction factors for creep are not required. Creep testing and evaluation should be conducted when the loading conditions and backfill and reinforcement conditions prescribed by FHWA (1) are exceeded.

The wall facing is composed of precast modular concrete blocks, which have a height of 8-inches. These types of blocks are readily available and need to conform to the same physical and chemical requirements as WisDOT MSE Wall Modular Blocks.

Special consideration should be given to the degree of batter of the various facing block systems. The amount of batter integrated into the wall systems can vary between manufacturers. Batter that is greater than expected will result in a decreased width between abutments when the span distance is held constant. The designer should be familiar with typical batter ranges for suppliers, and plan for variations in batter.

The wall facing blocks only support their self-weight and are held in place by the friction generated from their self-weight, the mechanical block interlocks, and the geotextile reinforcing fabric placed between each block layer. The upper layers of block will be less stable than the lower layers and they should be bonded in accordance with the specifications. This prevents movement of the blocks from expansion and contraction, freeze-thaw forces, settlement forces and vandalism.

The backfill should be an open graded material with an assumed internal angle of friction of 38 degrees. Generally this will limit the material to a crushed aggregate product. The RSF and integrated approach should generally use a wrapped dense graded aggregate.

The RSF and GRS mass should utilize a biaxial woven geotextile reinforcement fabric from the same manufacturer and of the same type and strength. Using biaxial geotextiles reduces the possibility of construction placement errors.

7.1.4.2.3.3 Superstructure

Typically, the bridge superstructure is placed directly on the reinforced soil abutment. Prestressed girders are often placed on top of the GRS substructure, followed by a traditional cast-in-place deck or precast deck panels. Other methods include the use of a cast-in-place concrete slab capable of spanning between the abutments or precast box girders. Both of these superstructure alternatives should be placed directly on the GRS abutment. The bearing area should contain additional geotextile reinforcement layers, which ensures that the superstructure bears on the GRS mass and not the facing blocks. The clear space between the facing block and the superstructure should be a minimum of 3-inches or 2 percent of the wall height, whichever is greater.

If steel or concrete I-girders are used, a precast or cast-in-place beam seat should be used to help distribute the girder reactions to the GRS abutment. Since there is open space between

I-girders, the beam seat can be used to support a backwall between the girders to retain the soil behind the girder ends.

7.1.4.2.3.4 Approach Integration

The approach construction that ties the roadway to the superstructure is essential for minimizing approach settlement and minimizing the bump at each end of the bridge. With a GRS abutment, this is accomplished by compacting and reinforcing the approach fill in wrapped geotextile layers and blending the integration zone with the approach pavement structure.

The integrated approach is constructed in a similar manner as the GRS mass, using layers of geotextile reinforcement and aggregate backfill. However, the integrated approach uses thinner layers until approximately 2 inches from the bottom of the pavement structure. The lift thicknesses should not exceed 6-inches and should be adjusted to accommodate the beam depths.

7.1.4.2.3.5 Design Details

Many of the typical detailing requirements for traditional bridges are still required on GRS-IBS bridges such as railings, parapets, guardrail end treatments, and drainage. Steel posts should be used for guardrail systems within the GRS and integrated approach areas, which can more easily penetrate the layers of geotextile than timber posts.

Penetrations and disturbances through the geotextile layers should be kept to a minimum and only used when absolutely necessary. Planning the locations of utilities and future utilities should be considered to avoid disturbing these layers. If utilities must be installed through a GRS-IBS abutment, all affected layers of geotextile should be overlapped/spliced according to the manufacturer’s recommendations.

The backfill used for GRS-IBS is usually comprised of free draining, open graded material. The designer should give consideration to providing additional drainage if warranted. Surface drainage should be directed away from the wall face and the reinforced soil mass.

7.1.4.2.4 Design Steps

The design of GRS-IBS abutments should follow a systematic process and is summarized below:

1. Establish Project Requirements
    - Determine geometry of abutment and wing walls (height, length, batter, back slope and toe slope, skew, grade, superelevation)
    - Ensure construction requirements are reasonable and economical
    - Determine the loading conditions (soil surcharge, dead load, live load, impact load, load from adjacent structures)
    - Determine performance criteria (tolerable settlements, displacements, and distortions, design life, constraints)

2. Perform a Site Evaluation
   - Study the existing topography
   - Check any existing structures/roads for problems
   - Conduct a subsurface investigation (foundation soil properties, groundwater conditions)
   - Evaluate soil properties for retained earth and reinforced backfill
   - Evaluate foundation soil properties to determine if shallow foundations are feasible at the site
   - Evaluate hydraulic conditions
   - Evaluate scour conditions to ensure shallow foundations are feasible at the site

3. Determine Layout of GRS-IBS
   - Define the geometry of the abutment face wall and wing walls
   - Lay out the abutment with respect to the superstructure (skew, superelevations, grade)
   - Account for setback and clear space to calculate the elevation of the abutment face wall and the span length of the bridge
   - Determine the depth and volume of excavation necessary for construction. A GRS abutment can be built with a truncated base to reduce the excavation. Truncation also reduces the requirements for backfill and reinforcement.
   - Determine the length of the reinforcement for the abutment
   - Add a bearing reinforcement zone underneath the bridge seat to support the increased loads due to the bridge
   - Blend the reinforcement layers in the integration zone to create a smooth transition

4. Calculate Applicable Loads
   - Lateral Pressures and Stresses

   - Dead Loads
     - Adjacent box beams can have the superstructure bearing directly on the GRS abutment
     - Dead load pressure includes bridge beams, overlay, railing, and any other applicable permanent loads related to the superstructure
   - Live Loads
   - Design Pressure

     Adding LL on the superstructure and the bridge DL per abutment will give the total load that the bridge seat must support. Dividing this total load by the area of the bridge seat will give the bearing pressure. For abutment applications, the bearing pressure should be targeted to approximately 4,000 lbs/ft^2. If this is exceeded, the width of the bridge seat should be increased.

5. Conduct an External Stability Analysis [If requirements not met, go back to Step 3]
   - Direct Sliding
   - Bearing Capacity
   - Global Stability

6. Conduct Internal Stability Analysis [If requirements not met, go back to Step 3]
    - Ultimate Capacity
    - Deformations
    - Required Reinforcement Strength

7. Implement Design Details
    - Conduct a hydraulic analysis (if necessary)
    - Ensure face of the abutment is wide enough to accommodate guardrail installation, including enough length for guardrail to lie down. Consider using native soil behind the reinforced backfill material at the abutment and two adjacent wing walls.
    - Determine whether to build wing walls with either a full face or a stepped face that leads into the cut slope
    - Check special requirements for skew, superelevation, and grade
    - Determine necessary construction compaction requirements and density testing methods for GRS and RSF granular backfill materials
    - Contain the GRS integrated approach fill by wrapping the geotextile layers adjacent to the beam ends to prevent lateral spreading
    - Avoid any abrupt transition of soil type from the roadway to the bridge
    - Locate and plan to accommodate existing and potential future utilities

7.1.4.3 Lateral Sliding

Bridge placement using lateral sliding is another type of ABC where the entire superstructure is constructed in a temporary location and is moved into place over a night or weekend. This method is typically used for bridge replacement of a primary roadway where the new superstructure is constructed on temporary supports adjacent and parallel to the bridge being replaced. Once the superstructure is fully constructed, the existing bridge structure is demolished, and the new bridge is moved transversely into place. In some instances, a more complicated method known as a bridge launch has been used, which involves longitudinally moving a bridge into place.

Figure 7.1-6
Lateral Sliding

Several different methods have been used to slide a bridge into place. One common method is to push the bridge using a hydraulic ram while the bridge slides on a smooth surface and Teflon coated elastomeric bearing pads. Other methods have also been used, such as using rollers instead of sliding pads, and winches in place of a hydraulic ram. The bridge can also be built on a temporary support frame equipped with rails and pushed or pulled into place along those rails. Many DOTs have successfully replaced bridges overnight using lateral sliding.

This ABC method is used to replace bridges that are part of a main transportation artery traversing a minor road, waterway, or other geographic feature. The limiting factor with using lateral slide is having sufficient right-of-way, and space adjacent to the existing bridge to construct the new superstructure.

7.1.4.4 ABC Using Self Propelled Modular Transporter (SPMT)

7.1.4.4.1 Introduction

SPMTs are remote-controlled, self-leveling (each axle has its own hydraulic cylinder), multi-axle platform vehicles capable of transporting several thousand tons of weight. SPMTs have the ability to move laterally, rotate 360° with carousel steering, and typically have a jack stroke of 18 to 24 inches. They have traditionally been used to move heavy equipment that is too large for standard trucks to carry. SPMTs have been used for bridge placement in Europe for more than 30 years. Over the past decade, the United States has implemented SPMTs for rapid bridge replacement following the FHWA's recommendation in 2004 to learn how other countries have used prefabricated bridge components to minimize traffic disruption, improve work zone safety, reduce environmental impact, improve constructability, enhance quality, and lower life-cycle costs. The benefits of ABC using SPMTs include the following:

1. Minimize traffic disruption: Building or replacing a bridge using traditional construction methods can require the bridge to be closed for months to years, with lane restrictions, crossovers, and traffic slowing for the duration of the closure. Using SPMTs, a bridge can be placed in a matter of hours, usually requiring only a single night or weekend of full road closure and traffic divergence.

2. Improve work zone safety: The bridge superstructure is constructed in an off-site location called a bridge staging area (BSA). This allows construction of the entire superstructure away from live traffic, which improves the safety of both the construction workers and the traveling public.

3. Improve constructability: The BSA typically offers better construction access than traditional construction by keeping workspaces away from live traffic, environmentally sensitive areas, and over existing roadways.

4. Enhance quality: Bridge construction takes place off-site at the BSA where conditions can be more easily controlled, resulting in a better product. There is an opportunity to provide optimal concrete cure time in the BSA because the roadway in the temporary location does not have traffic pressures to open early.

5. Lower life-cycle costs: Because the quality of the bridge is increased, the overall durability and life of the bridge is also increased. This reduces the life-cycle cost of the structure.

6. Provide opportunities to include other ABC technologies: Multiple ABC technologies can be used on the same project, for example, a project could utilize prefabricated bridge elements, and also be moved into place using SPMTs.

7. Reduce environmental impacts: SPMT bridge moves have significantly shorter on-site construction durations than traditional construction, which is particularly advantageous for areas that are environmentally sensitive. These areas may restrict on-site construction durations due to noise, light, or night work.

Figure 7.1-7

Self Propelled Modular Transporters Moving a Bridge

When replacing a bridge using SPMTs the new superstructure is built on temporary supports off-site in a designated BSA near the bridge site. Once the new superstructure is constructed, the existing structure can be removed quickly with SPMTs or can be demolished in conventional time frames, depending on the project-specific needs. Once the existing structure is removed, the new superstructure is moved from the staging area to the final location using two or more lines of SPMT units. The SPMTs lift the superstructure off of the temporary abutments and transport it to the permanent substructure. The placement of a bridge superstructure using SPMTs often requires only one night of full road closure, and many bridges in the United States have been placed successfully in a matter of hours.

When using SPMTs for bridge replacement a new substructure may be constructed, or the existing substructure may be reused. If the existing substructure is in good condition and meets current design requirements, it may be reused, or it may be rehabilitated. When constructing a new substructure, the new abutments are often built below the superstructure in front of the existing abutments, so the construction can advance before deconstruction of the existing structure begins. Because the superstructure is constructed in the BSA, the new superstructure can be constructed at the same time as the substructure.

SPMTs are typically used to replace bridges that carry or span major roadways. Time limitations or impacts to traffic govern the need for a quick replacement. Locating an off-site BSA to build the superstructure is a critical component for using SPMTs. There needs to be a clearly defined travel path (TP) between the staging area and the final bridge location that can support the SPMT movements (vertical clearances, horizontal clearances, turning radii, soil conditions, utility conflicts, etc.). See sections 7.1.4.4.6.1 and 7.1.4.4.6.2 for additional discussion of the BSA and TP.

SPMTs can also be used to place a bridge over a waterway. In this case, the bridge superstructure is constructed offsite, and then SPMTs transport the superstructure from the BSA onto a barge which travels the waterway to the final bridge site.

To date, mostly single-span bridges or individual spans of multi-span bridges with lengths ranging from approximately 100 to 200 feet have been moved with SPMTs. There have been a few two-span bridge moves with SPMTs in the United States. The most common structures that have been moved successfully are prestressed I-girder or steel plate girder bridges.

The following sections discuss key items for bridge placement using SPMT in the State of Wisconsin. For additional information on the use of SPMTs for the movement of bridges consult FHWA's *Manual on Use of Self Propelled Modular Transporters to Remove and Replace Bridges*, and UDOT's *SPMT Manual*. Contact the WisDOT Bureau of Structures Design Section as an additional resource.

7.1.4.4.2 Application

For guidance on whether SPMT bridge placement or another ABC technology should be used for a project, first refer to the WisDOT ABC decision making guidance spreadsheet and flowchart in Section 7.2. Some of the common criteria that govern the use of SPMTs are the following:

1. There is a need to minimize the out-of-service window for the roadway(s) on or under the structure

2. There is a major railroad track on or under the bridge

3. There is a major navigation channel under the bridge

4. The bridge is an emergency replacement

5. The road on or under the bridge has a high ADT and/or ADTT

6. There are no good alternatives for staged construction or detours

7. There is a sensitive environmental issue

Along with the use of this technology, the specifications need to include incentives and disincentives to employ for the project.

7.1.4.4.3 Special Provision

When writing a special provision for a project using SPMTs, consider the following items that may need to be included in the special provision text:

1. Drainage – Define areas (bridge site, BSA, TP, etc.) where drainage needs to be maintained throughout construction and indicate areas where temporary culvert pipes will be required. In the special provision text, clearly indicate if the temporary culvert pipes are to be included with the “SPMT Bridge Construction B-XX-XXX”.

2. Temporary Concrete Barrier – define areas where temporary concrete barrier is required. Clearly indicate which barriers (temporary or permanent) are paid for with the roadway bid items, and which barriers are paid for with the item “SPMT Bridge Construction B-XX-XXX”.

3. Bearing Pads – Indicate if bearing pads need to be adhered to the bottoms of girders prior to the bridge move or if temporary bearing pads are required on the temporary supports. Clearly indicate how the bearing pads are to be paid.

7.1.4.4.4 Roles and Responsibilities

The following sections outline the roles and responsibilities for the parties involved in the project using the design-bid-build delivery method. These roles apply if WisDOT specifies that the bridge will be placed using SPMTs. If SPMT use is not a stated requirement for the project, the Contractor may have the option to use them as long as the project specifications are met. If this occurs, the contractor would assume the responsibilities for certain items in Table 7.1-2 as described in 7.1.4.4.4.3.

Category	Responsibility Description	Responsible Party
Scoping	Decision to Use SPMTs	WisDOT Region & BOS
	Bridge Type Selection	Designer
	Provide Resources to Design Team	WisDOT BOS
Superstructure	Superstructure Design	Designer
Pick Points	Location and Tolerances	Designer
	Analyze Bridge for Effects from Lifting and Travel	Designer
Deflections	Set Stress, Deflection, and Twist Limits	WisDOT & Designer
	Monitoring Plan (Specifications)	Designer
	Monitoring Plan (Execution)	Contractor
BSA and TP	Location of BSA	Designer
	Geometry of TP	Designer
Utilities	Utility Agreements	WisDOT
	Mitigation Concepts	Designer
	Mitigation Execution	Contractor
Site Conditions	Structural Analysis of Bridge Along TP	Designer
	Set Allowable Stress Limits on BSA and TP	Designer
	Mitigation of Affected Areas at BSA and TP	Contractor
	Protection of Structure Along TP	Contractor
Heavy Lifter Equipment	SPMT	Contractor
	Heavy Lifter Equipment to Raise Bridge	Contractor
	Contingency Plan For Equipment Failure	Contractor
Support Structures	Permanent Substructure Design	Designer
	Temporary Support Design	Contractor

Table 7.1-2
SPMT Roles and Responsibilities

7.1.4.4.4.1 WisDOT

The WisDOT Region and the Bureau of Structures shall make the final decision to use SPMTs on a project, considering user costs. WisDOT either specifies to the designer that SPMTs will be used for the project, or they allow the contractor to propose an ABC method. If the latter is chosen, the project parameters, specification, schedule, and proposal should be defined in a way that ensures the requirements are met if the contractor decides that an SPMT move is the best solution.

7.1.4.4.4.2 Designer

The Designer includes any traffic, structural, or geotechnical engineers engaged by WisDOT in the design of the project. Final drawings and calculations should be stamped by a Professional Engineer licensed in the State of Wisconsin. The permanent substructure and superstructure should be designed in accordance with AASHTO LRFD Specifications and WisDOT Bridge Manual requirements. The superstructure should be designed to withstand induced forces from lifting off of temporary supports, transportation along TP, and lowering onto permanent bearings.

The Designer determines the feasibility of a BSA and TP, considering the following items at a minimum: geotechnical concerns, conflicting utilities, real estate and conflicting obstacles. The Designer also specifies the monitoring plan and maximum bearing pressure along travel path.

The Designer should deliver a project that can accommodate travel conditions during transportation of the structure on the SPMT units. Braking forces while the bridge is on the SPMTs shall be accounted for. Consider placing diaphragms at the pick points for additional lateral support.

7.1.4.4.4.3 Contractor

The Contractor may include the General Contractor, Heavy Lifter or SPMT Contractor, any bridge specialty engineers, and/or any other subcontractor employed by the General Contractor for the construction of the project.

The Contractor is responsible for:

1. The design of all temporary structures.
2. The construction of all structures, permanent or otherwise.
3. The design of the support system between the SPMT units and the bridge at final position.
4. The redesign and changes to plans to adjust for constructability issues based on the transport system chosen.
5. The design of the blocking or structure that supports the bridge during transport.
6. The safe transport of the bridge from the BSA to the final bridge location, ensuring that no maximum stresses or deflections are exceeded.

The Contractor is required to:

1. Provide all required plans, calculations, etc. in accordance with the specifications.
2. Identify, design and implement any required ground improvements in the BSA and TP.

3. Provide a contingency plan in the case of equipment malfunction or failure.

If the Contractor requests and is granted departmental approval to use SPMTs on a project that has not been designed for SPMT use, the following responsibilities (Refer to Table 7.1-2) that others are typically responsible for would be assumed by the Contractor:

1. Utilities – Mitigation Concepts

2. Site Conditions – Structural Analysis of Bridge Along TP

3. Site Conditions – Set allowable stress limits on BSA and TP

4. All Items under the category of Pick Points, Deflections (analysis), BSA and TP

5. Acquiring real estate

7.1.4.4.5 Temporary Supports

Temporary supports include temporary shoring and abutments that support the superstructure in the BSA and on the SPMTs during transport. The contractor is responsible for the design and construction of temporary supports. Temporary structures should be designed using *AASHTO Guide Design Specifications for Bridge Temporary Works*.

Design the temporary supports in the BSA to withstand a minimum lateral load equal to 10% of the superstructure dead load. Other lateral loads, such as wind, need not be included with this loading scenario.

These structures should provide bearing support conditions similar to the permanent bearings. The bridge superstructure is typically constructed in the temporary location with the same vertical clearance under the structure as the permanent location. The bridge may be constructed at a lower elevation for ease of construction; however this requires jacking the superstructure up to the correct elevation prior to transport.

SPMT blocking is the temporary support during transport that supports the superstructure at the pick point and connects to the SPMT units. Design SPMT blocking to withstand the forces induced during transport such as braking, turning, elevation changes, and wind loads.

7.1.4.4.6 Design Considerations

7.1.4.4.6.1 Bridge Staging Area

The BSA is the temporary location where the bridge superstructure will be constructed. The BSA is an area within the right of way, an offsite location, or an area acquired by the contractor. If an existing bridge is being removed using SPMTs, the BSA should provide adequate space for the superstructure to be removed. For projects with multiple bridges or one bridge with multiple simple spans, one or more bridges may occupy a single BSA. Figure 7.1-8 shows an example BSA that accommodated several structures.

Figure 7.1-8
Example Bridge Staging Area (BSA)

The BSA soil must have enough capacity to support the SPMTs carrying the superstructure. This requires a geotechnical investigation of the soils with possible additional measures such as ground improvements, soft soil mitigation, and utility protection. The contractor may need to address the bearing capacity of the soil in different manners based on the particular SPMT equipment that is selected. The BSA must be clear of all obstacles during bridge construction.

The designer specifies the maximum soil pressure in the BSA and TP based on the actual weight of the structure, anticipated SPMT weight, and temporary blocking. SPMT and temporary blocking weights need to be assumed. The design shall include a 5% dead load increase to cover miscellaneous loads (concrete tolerances, miscellaneous items, equipment during the move, etc.).

7.1.4.4.6.2 Travel Path

The TP is the path that the SPMTs use to transport the bridge(s) from the BSA to the final bridge location. The TP has similar requirements as the BSA. A geotechnical investigation is required to determine the need for ground improvements, soft soil mitigation, and utility protection. Steel plates, spreader beams, temporary pavement, and soft soil replacement are different methods used to help distribute the load and control settlement over these sensitive areas. Even a small area of soft soil can be detrimental during a superstructure transport. If the soil collapses under an SPMT tire, it can be extremely difficult to continue the bridge transport.

SPMT units are capable of traveling on uneven surfaces, however, it is preferred to keep the surface of the TP as level as possible with gradual elevation changes to minimize deflection and twist in the superstructure. Contact the WisDOT Bureau of Structures Design Section for approval of an uneven TP surface.

7.1.4.4.6.3 Allowable Stresses

During the process of lifting, transporting, and placing a bridge using SPMTs, the superstructure will undergo stresses different than those induced with traditional cast in place bridge construction. These stresses include stress reversals as described in 7.1.4.4.6.4. For calculation of the stresses in the superstructure when supported on the SPMTs, an impact factor of 1.15 applied to the dead load shall be used.

The Designer calculates the allowable stresses in the deck and in the girders. The bridge should be designed so that the reinforcement in the deck and parapet will not yield during transport of the bridge.

7.1.4.4.6.4 Pick Points

Pick points are the bearing locations where the superstructure is lifted off the temporary supports by the SPMTs and transported to the permanent location. Pick points should be located within 20% of the span length from the ends of the superstructure. This minimizes the cantilevered portion and negative forces induced on the superstructure. During the lifting of the superstructure off the temporary supports, the bridge undergoes a stress reversal. When the girders are placed and the deck is poured, the girders deflect under the wet concrete weight, inducing stresses in the girder. When the deck is cured, the stresses in the girders induced by the deck are locked in, and the superstructure is in a state of equilibrium. Changing the support locations causes a stress reversal in the superstructure, which must be considered in the design of the bridge.

Figure 7.1-9 illustrates the stress reversal that the superstructure undergoes when the bearing locations are changed. The easiest way to visualize this change is through the moment diagrams in the figure. The first diagram in the figure illustrates the moment on the superstructure due to dead loads with the support system at the ends similar to the final bearing system. The moment, M_a, is the moment at the pick point location. The second moment diagram shows the moments when the superstructure is supported at the pick points. Again, the moment, M_b, is the moment at the pick point location. The third diagram in the figure shows the two moments superimposed. The total stress that the superstructure sees at the pick point location, M_c, is from the two moments combined. Please note that this illustration is very simplified, and more in depth calculations and/or finite element modeling is required in order to calculate the actual stresses on the deck.

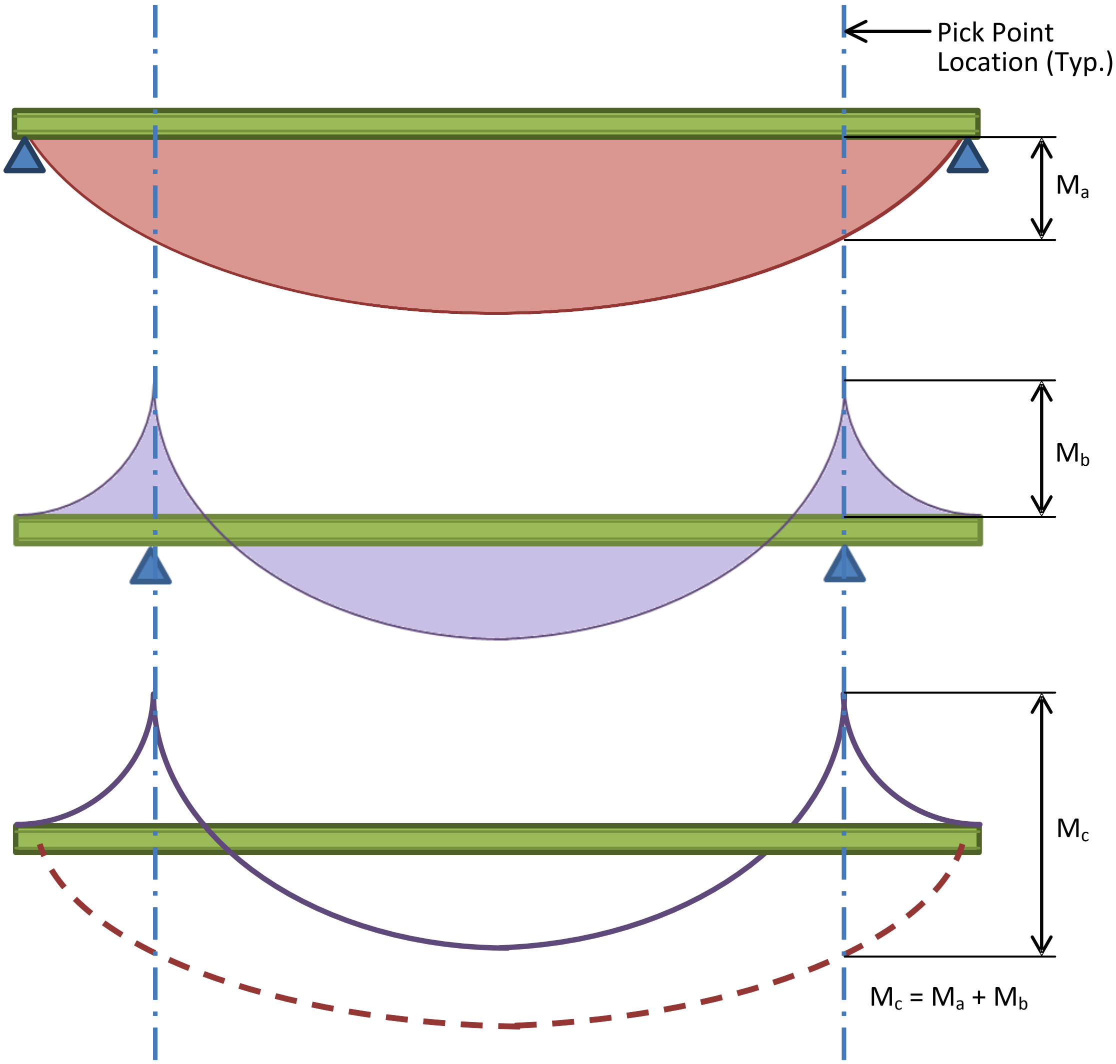

Figure 7.1-9
Support Change Moment Diagram (Illustrating Stress Reversal)

The construction sequence also complicates stress considerations. In the construction sequence, the girders are placed and the concrete is poured for the deck. The deck cures with essentially no stress, but the stress in the girders due to the deck pour is locked in when the girder and deck become composite. When the SPMTs engage the superstructure at the pick points, the girders go from positive bending at the pick points to negative bending. The deck at the pick point locations transitions from a state of zero bending (zero stress) to a state of negative bending. The stress calculations for the deck will be based on the composite moment of inertia.

The pick points must be located on the bridge in a manner to limit the tension in the deck. Clearly show pick points in the plans, and ensure that stresses induced from lifting and transporting the superstructure are within the allowable stresses shown in plans.

7.1.4.4.6.5 Deflection and Twist

During transport of the bridge from the BSA to its final position, the bridge will deflect and twist. Minor deflection and twist is to be expected during the movement of the bridge, but excessive deflections induce unwanted stresses in the deck that can cause cracking or other permanent damage to the superstructure. The bridge should be monitored during transport to keep the deflection and twist within specified limits. The specifications should outline the allowable deflections for the specific circumstances and structure(s). A critical point in the movement of the bridge is when the bridge is initially lifted off of the temporary supports. The stress reversal discussed in 7.1.4.4.6.4 will occur during this initial lift.

Warping and/or twisting of the bridge occurs when uneven bearing supports cause the slope of the bearing lines to be different from each other at each end of the span. Figure 7.1-10 shows an illustration of bridge warping. The blue solid square shows the as-constructed plane of the bridge. The red lines show the warped bridge plane and the dashed red lines represent the relative deflection from the as-constructed position.

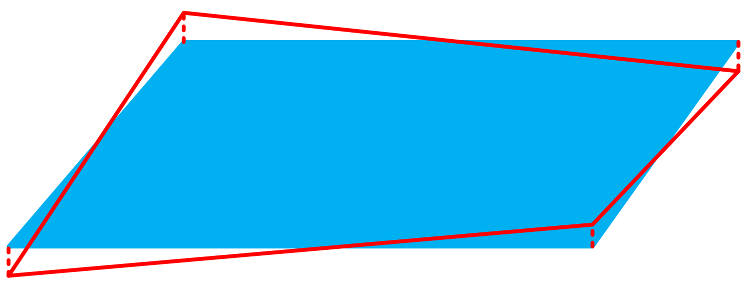

Figure 7.1-10
Bridge Warping Diagram

A monitoring plan should be developed by the Designer to monitor deflection and twist of the superstructure. Survey of critical points should be taken after construction of the superstructure and immediately after lifting it off of the temporary supports. A system should be established to monitor the relative deflections of each corner of the bridge during the transportation of the bridge. An example of bridge monitoring for deflection and twist can be found in UDOT's *Manual for the Moving of Utah Bridges Using Self Propelled Modular Transporters (SPMTs)*.

Accurate deflection calculations are very important when considering the SPMT unit jack stroke. For example, if the superstructure needs to be jacked 6 inches in order to lift the bridge off the temporary supports at the pick points, one quarter of the SPMT jack stroke would be used solely to lift the superstructure (assuming a typical jack stroke maximum of 24 inches).

Figure 7.1-11 illustrates how the deflection is accounted for in raising the superstructure off the temporary supports. Deflection, Δ_a, is the dead load deflection of the superstructure at the pick point location relative to the ends when the bridge is supported at the ends. Deflection, Δ_b, is the dead load deflection of the composite structure between the pick point location and the end support location when the bridge is supported at the pick point locations. Deflection, Δ_c, is the distance required to raise the structure off the temporary support.

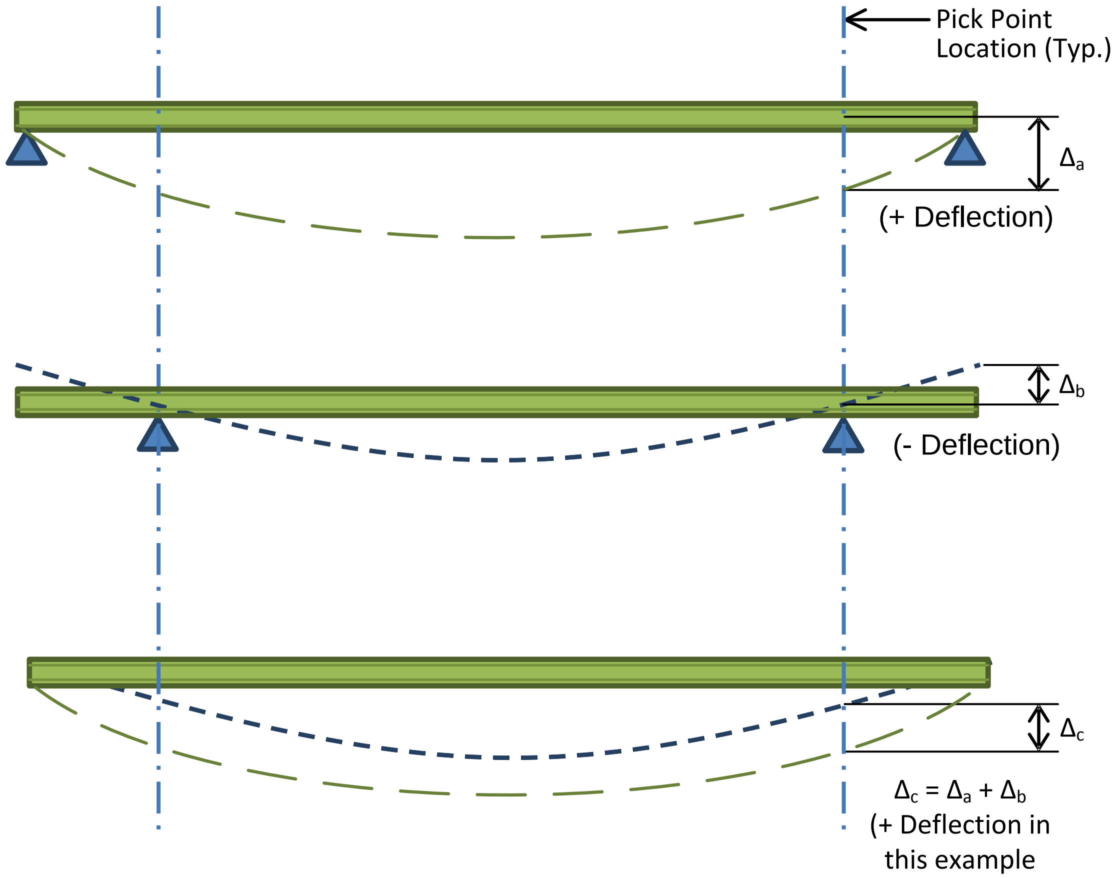

Figure 7.1-11
Support Change Deflection Diagram

Note: For this example, assume positive deflections are downward.

7.1.4.4.7 Structure Removal Using SPMT

When using SPMTs for bridge replacement, an alternative to onsite demolition of the existing bridge superstructure is removing the bridge using SPMTs. The existing superstructure can be removed and transported to the BSA where it is placed on temporary abutments until it can be demolished or salvaged. This method eliminates the need for protection of the underlying roadway and substructure elements.

All TP and BSA considerations, covered in 7.1.4.4.6.2 and 7.1.4.4.6.1 respectively, must be addressed for the movement of the existing superstructure. Follow guidelines in 7.1.4.4.5 for the design of temporary supports for existing superstructure.

7.1.5 Project Delivery Methods/Bidding Process

In addition to the accelerating technologies discussed in this chapter, the Every Day Counts initiative includes accelerated project delivery methods as a way to shorten the project duration. Traditionally, the Design-Bid-Build (DBB) method has been used for project delivery. This involves the design and construction to be completed by two different entities. Project schedules using the DBB method are elongated because the design and construction cannot be completed concurrently. The entire design process must be completed before the bidding process begins. Finally, after the bidding process is completed, the construction can begin.

Other state DOT's have used project delivery methods that can allow for more accelerated overall project delivery. These include Design/Build (D/B) and Construction Manager/General Contractor (CM/GC). The D/B process requires the designer-builder to assume responsibility for both the design and construction of the project. This method increases the risk for the design-builder, and reduces the risk for the owner. Project delivery time can be reduced, since the D/B process allows for the design and construction phases to overlap, unlike the DBB process. There is a specific type of D/B called Low Bid Design Build (LBDB) which has the same structure as the traditional D/B process, except that the lowest bidder wins the project (rather than having a quality component as with the traditional D/B process). Refer to the *Facilities Development Manual (FDM)* for further discussion on LBDB.

The CM/GC process is a hybrid of the DBB and D/B processes. In CM/GC, both the designer and the contractor have contracts with the owner, and the owner is part of the design team. In this process, a construction manager is selected, and is able to provide input regarding schedule, pricing, and phasing during the design phase. Around the 60% or 90% design completion, the owner and construction manager negotiate a "guaranteed maximum price" for the construction of the project based on the defined scope and schedule. CM/GC allows the owner to remain active in the design process, while the risk is still taken by the general contractor.

Generally, in Wisconsin, projects administered by the Department have been Design Bid Build with minimal use of the Low Bid Design Build method. Refer to the FDM Chapter 11-50-32 for additional discussion on Alternative Contracting (AC) methods.

WisDOT policy item:

Each state has different preferences and constraints to which project delivery method they use, and due to current legislation, CM/GC and traditional D/B are not viable options for the state of Wisconsin. To implement ABC using the DBB process, the contract should either specify to use the ABC method required by the owner, and/or provide opportunity for the contractor to propose ABC alternatives that meet contract requirements.

7.2 ABC Decision-Making Guidance

This section is intended to provide guidance on when to use ABC versus conventional construction. When ABC methods are appropriate, this section will also help determine which ABC method(s) are most practical for a particular project.

Figure 7.2-1 is a Decision Matrix that can be used to determine how applicable an ABC method is for a particular project. Each item in Figure 7.2-1 is described further in Table 7.2-1. Once a total score is obtained from the Decision Matrix, the score is used to enter the Decision Flowchart (Figure 7.2-2). After entering the Flowchart, the user could be directed to the question "Do the benefits of ABC outweigh any additional costs?" This question needs to be evaluated on a project-specific basis, using available project information and engineering judgment. This item is intended to force the user to step back, think about the project as a whole, and decide if an ABC method really makes sense with all the project-specific information considered. The remainder of the flow chart questions will help guide the user toward the ABC method(s) that are most appropriate for the project.

There is an acknowledged level of subjectivity in both the Decision Matrix and in the Flowchart. These tools are intended to provide general guidance, not to provide a specific answer for all projects. The tools present different types of considerations that should be taken into account to help guide the user in the right direction and are not intended to provide a "black and white" answer.

The flowchart item "Program Initiative" can encompass a variety of initiatives, including (but not limited to) research needs, public input, local initiatives, stakeholder requests, or structure showcases. These items should be considered on a project-specific basis.

The flowchart guides users towards specific ABC technologies. However, the user should also recognize the ability and opportunity to combine various ABC technologies. For example, the combination of PBES with GRS-IBS could be utilized.

For additional guidance or questions, contact the Bureau of Structures Development Section Chief.

% Weight	Category	Decision-Making Item	Possible Points	Points Allocated	Scoring Guidance
17%	Disruptions (on/under Bridge)	**Railroad on Bridge?**	8		0 No railroad track on bridge 4 Minor railroad track on bridge 8 Major railroad track on bridge
		Railroad under Bridge?	3		0 No railroad track under bridge 1 Minor railroad track under bridge 3 Major railroad track(s) under Bridge
		Over Navigation Channel that needs to remain open?	6		0 No navigation channel that needs to remain open 3 Minor navigation channel that needs to remain open 6 Major navigation channel that needs to remain open
8%	Urgency	**Emergency Replacement?**	8		0 Not emergency replacement 4 Emergency replacement on minor roadway 8 Emergency replacement on major roadway
23%	User Costs and Delays	**ADT and/or ADTT** (Combined Construction Year ADT on and under bridge)	6		0 No traffic impacts 1 ADT under 10,000 2 ADT 10,000 to 25,000 3 ADT 25,000 to 50,000 4 ADT 50,000 to 75,000 5 ADT 75,000 to 100,000 6 ADT 100,000+
		Required Lane Closures/Detours? (Length of Delay to Traveling Public)	6		0 Delay 0-5 minutes 1 Delay 5-15 minutes 2 Delay 15-25 minutes 3 Delay 25-35 minutes 4 Delay 35-45 minutes 5 Delay 45-55 minutes 6 Delay 55+ minutes
		Are only Short Term Closures Allowable?	5		0 Alternatives available for staged construction 3 Alternatives available for staged construction, but undesirable 5 No alternatives available for staged construction
		Impact to Economy (Local business access, impact to manufacturing etc.)	6		0 Minor or no impact to economy 3 Moderate impact to economy 6 Major impact to economy
14%	Construction Time	**Impacts Critical Path of the Total Project?**	6		0 Minor or no impact to critical path of the total project 3 Moderate impact to critical path of the total project 6 Major impact to critical path of the total project
		Restricted Construction Time (Environmental schedules, Economic Impact – e.g. local business access, Holiday schedules, special events, etc.)	8		0 No construction time restrictions 3 Minor construction time restrictions 6 Moderate construction time restrictions 8 Major construction time restrictions
5%	Environment	**Does ABC mitigate a critical environmental impact or sensitive environmental issue?**	5		0 ABC does not mitigate an environmental issue 2 ABC mitigates a minor environmental issue 3 ABC mitigates several minor environmental issues 4 ABC mitigates a major environmental issue 5 ABC mitigates several major environmental issues
3%	Cost	**Compare *Comprehensive* Construction Costs** (Compare conventional vs. prefabrication)	3		0 ABC costs are 25%+ higher than conventional costs 1 ABC costs are 1% to 25% higher than conventional costs 2 ABC costs are equal to conventional costs 3 ABC costs are lower than conventional costs
18%	Risk Management	**Does ABC allow management of a particular risk?**	6		0 - 6 Use judgment to determine if risks can be managed through ABC that aren't covered in other topics
		Safety (Worker Concerns)	6		0 Short duration impact with TMP Type 1 3 Normal duration impact with TMP Type 2 6 Extended duration impact with TMP Type 3-4
		Safety (Traveling Public Concerns)	6		0 Short duration impact with TMP Type 1 3 Normal duration impact with TMP Type 2 6 Extended duration impact with TMP Type 3-4
12%	Other	**Economy of Scale** (repetition of components in a bridge or bridges in a project) (Total spans = sum of all spans on all bridges on the project)	5		0 1 total span 1 2 total spans 2 3 total spans 3 4 total spans 4 5 total spans 5 6+ total spans
		Weather Limitations for conventional construction?	2		0 No weather limitations for conventional construction 1 Moderate limitations for conventional construction 2 Severe limitations for conventional construction
		Use of Typical Standard Details (Complexity)	5		0 No typical standard details will be used 3 Some typical standard details will be used 5 All typical standard details will be used
		Sum of Points:		0	(100 Possible Points)

Figure 7.2-1
ABC Decision-Making Matrix

7.2.1 Descriptions of Terms in ABC Decision-Making Matrix

The following text describes each item in the ABC Decision-Making Matrix (Figure 7.2-1). The points associated with the scoring guidance in the matrix and in the text below are simply *guidance*. Use engineering judgment and interpolate between the point ranges as necessary.

Decision-Making Item	Scoring Guidance Description
Railroad on Bridge?	This is a measure of how railroad traffic on the bridge will be affected by the project. If a major railroad line runs over the bridge that requires minimum closures and a shoo fly (a temporary railroad bridge bypass) cannot be used, provide a high score here. If a railroad line that is rarely used runs over the bridge, consider providing a mid-range or low score here. If there is no railroad on the bridge, assign a value of zero here.
Railroad under Bridge?	This is a measure of how railroad traffic under the bridge will be affected by the project. If a major railroad line runs under the bridge that would disrupt construction progress significantly, provide a high score here. If a railroad track runs under the structure, but it is used rarely enough that it will not disrupt construction progress significantly, provide a low score here. Consider if the railroad traffic is able to be suspended long enough to move a new bridge into place. If there is not a large enough window to move a new bridge into place, SPMT could be eliminated as an alternative for this project. For this case, PBES may be a more applicable alternative. If there is no railroad under the bridge, assign a value of zero here.
Over Navigation Channel that needs to remain open?	This is a measure of how a navigation channel under a bridge will be affected by the project. If a navigation channel is highly traveled and needs to remain open for shipments, provide a high score here. If a navigation channel is rarely traveled and there are not requirements for it to remain open at certain time periods, provide a low score here. If there is no navigation channel under the bridge, assign a value of zero here.
Emergency Replacement?	This is a measure of the urgency of the bridge replacement. A more urgent replacement supports the use of accelerated bridge construction methods, since demolition and construction can be progressing concurrently. Depending on the particular project, accelerated bridge construction methods can also allow multiple components of the bridge to be constructed concurrently. If the bridge replacement is extremely urgent and the bridge can be replaced quicker by using accelerated construction methods, provide a high score here.

ADT and/or ADTT (Construction Year)	This is a measure of the total amount of traffic crossing the bridge site. A higher ADT value at a site will help support the use of accelerated bridge construction methods. Use a construction year ADT value equal to the sum of the traffic on the structure and under the structure. For cases where there is a very high ADT on the bridge and very low or no ADT under the bridge, consider using a “slide” method (on rollers or Polytetrafluorethylene (PTFE)/Elastomeric pads) or SPMT’s, which can be very cost effective ABC techniques for this situation. For structures with a higher-than-average percentage of truck traffic, consider providing a higher score than indicated solely by the ADT values in the table.
Required Lane Closures/Detours?	This is a measure of the delay time imposed on the traveling public. If conventional construction methods will provide significant delays to the traveling public, provide a high score here. If conventional construction methods will provide minimal delays to the traveling public, provide a low score here. Use the delay times provided in the table as guidance for scoring.
Are only Short Term Closures Allowable?	This is a measure of what other alternatives are available besides accelerated bridge construction. If staged construction is not an alternative at a particular site, the only alternative may be to completely shut down the bridge for an SPMT move, and therefore a high score should be provided here. If there is a good alternative available for staged construction that works at the site, a low score should be provided here.
Impact to Economy	This is a measure of the impact to the local businesses around the project location. Consider how the construction staging, road closures, etc. will impact local businesses (public access, employee access, etc.) A high impact to the economy equates to a high score here. A low impact to the economy equates to a low score here.
Impacts Critical Path of Total Project?	This is a measure of how the construction schedule of the structure impacts the construction schedule of the entire project. If the construction of the structure impacts the critical path of the entire project, and utilizing ABC methods provides shorter overall project duration, provide a high score here. If other project factors are more critical for the overall project schedule and utilizing ABC methods will not affect the overall project duration, provide a low score here.
Restricted Construction Time	This is a measure of how the construction schedule is impacted by environmental and community concerns or requirements. Items to consider are local business access windows, holiday schedules and traffic, special event traffic, etc. If there are significant restrictions on construction schedule, provide a high score here. If there are little to no restrictions on the construction schedule, provide a low score here.

Does ABC mitigate a critical environmental impact or sensitive environmental issue?	This is a measure of how using accelerated bridge construction methods can help mitigate impacts to the environment surrounding the project. Since accelerated methods allow a shorter on-site construction time, the impacts to the environment can be reduced. If the reduced on-site construction time provided by accelerated bridge construction methods mitigates a significant or critical environmental concern or issue, provide a high score here. If there are no environmental concerns that can be mitigated with accelerated construction methods, provide a low score here.
Compare Comprehensive Construction Costs	This is a measure of the complete comprehensive cost difference between conventional construction methods versus using an accelerated bridge construction method. Some costs will increase with the use of accelerated construction methods, such as the cost of the SPMT equipment and the learning curve that will be incorporated into using new technologies. However, some costs will decrease with the use of accelerated construction methods, such as the reduced cost for traffic control, equipment rentals, inspector wages, etc. Many of the reduced costs are a direct result of completing the project in less time. Use the cost comparisons in the table as guidance for scoring here.
Does ABC allow management of a particular risk?	This is an opportunity to add any project-specific items or unique issues that have risk associated with them that are not incorporated into another section in this text. Consider how ABC may or may not manage those particular risks.
Safety (Worker Concerns)	This is a measure of the relative safety of the construction workers between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of workers in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here. Refer to the *Facilities Development Manual (FDM)* for definitions of TMP Types.
Safety (Traveling Public Concerns)	This is a measure of the relative safety of the traveling public between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of the traveling public in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here. Refer to the *Facilities Development Manual (FDM)* for definitions of TMP Types.

Economy of Scale	This is a measure of how much repetition is used for elements on the project, which can help keep costs down. Repetition can be used on both substructure and superstructure elements. To measure the economy of scale, sum the total number of spans that will be constructed on the project. For example, if there are 2 bridges on the project that each have 2 spans, the total number of spans on the project is equal to 4. Use the notes in the table for scoring guidance here.
Weather Limitations for Conventional Construction?	This is a measure of the restrictions that the local weather causes for on-site construction progress. Accelerated bridge construction methods may allow a large portion of the construction to be done in a controlled facility, which helps reduce delays caused by inclement weather (rain, snow, etc.). Depending on the location and the season, faster construction progress could be obtained by minimizing the on-site construction time.
Use of Typical Standard Details (Complexity)	This is a measure of the efficiency that can be gained by using standard details that have already been developed and approved. If standard details are used, some errors in the field can be prevented. If new details are going to be created for a project, the contractors will be less familiar with the details and problems may arise during construction that were not considered in the design phase. Use the notes in the table for scoring guidance here.

Table 7.2-1
ABC Decision-Making Matrix Terms

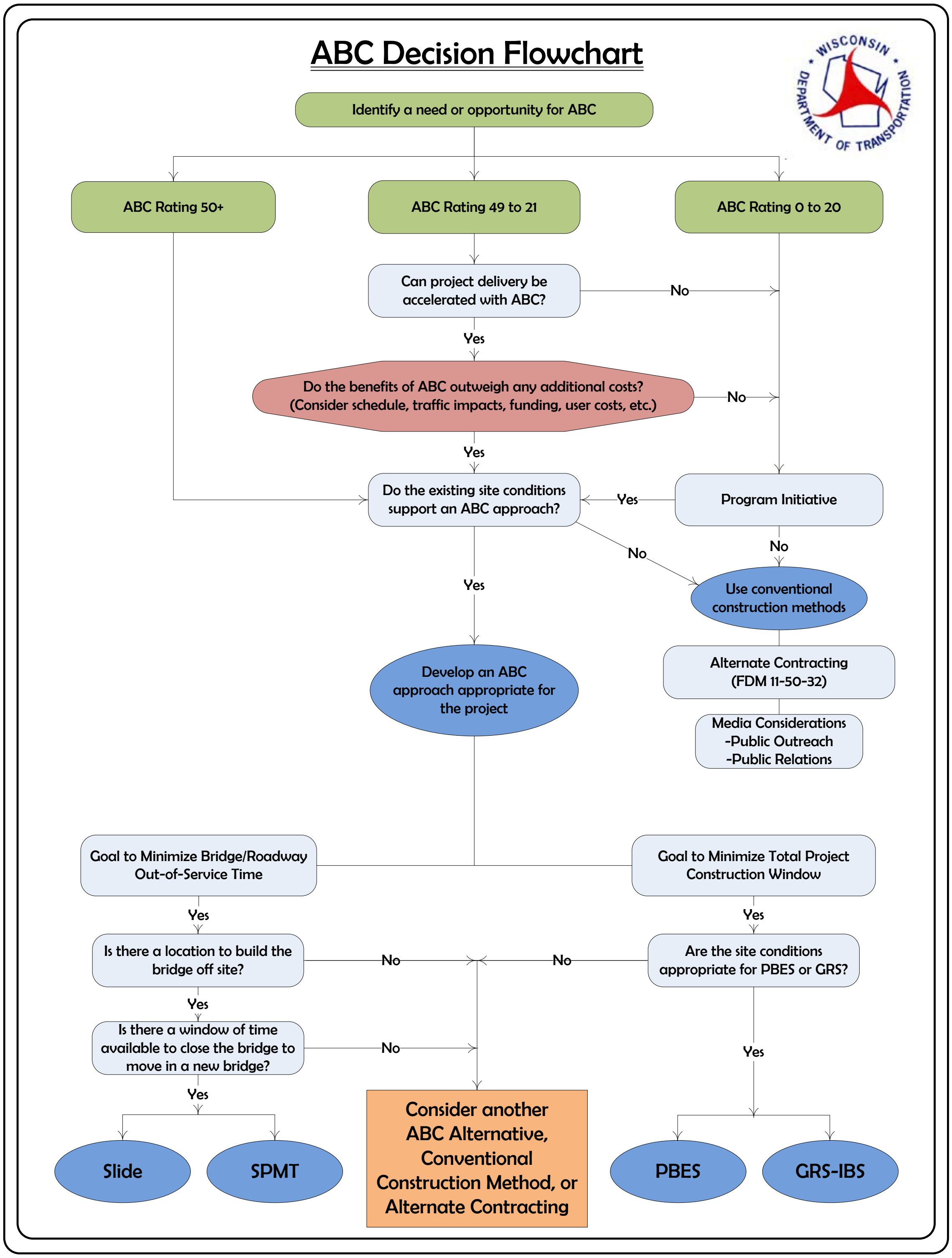

Figure 7.2-2
ABC Decision-Making Flowchart

7.3 References

1. Every Day Counts Initiative. Federal Highway Administration. 23 May. 2012. http://www.fhwa.dot.gov/everydaycounts/

2. Federal Highway Administration. Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide. U.S. Department of Transportation. McLean, VA: Turner-Fairbank Highway Research Center, 2011. FHWA-HRT-11-026

3. Federal Highway Administration. Geosynthetic Reinforced Soil Integrated Bridge System Synthesis Report. U.S. Department of Transportation. McLean, VA: Turner-Fairbank Highway Research Center, 2011. FHWA-HRT-11-027.

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Table of Contents

8.1 Introduction

The methods of hydrologic and hydraulic analysis provided in this chapter give the designer information necessary for an analysis of a roadway drainage crossing. Experience and sound engineering judgment are not to be ignored and may, at times, differ from results obtained using methods in this chapter. Very careful weighing of experience, judgment, and procedure must be made to arrive at a solution to the problem. Research in the field of drainage continues throughout the country and may subsequently alter the procedures found in this chapter.

8.1.1 Objectives of Highway Drainage

The objective of highway drainage is to prevent the accumulation and retention of water on and/or around the highway by:

- Anticipating the amount and frequency of storm runoff.
- Determining natural points of concentration of discharge and other hydraulic controls.
- Removing detrimental amounts of surface and subsurface water.
- Providing the most efficient hydraulic design consistent with economy, the importance of the road, maintenance and legal obligations.

8.1.2 Basic Policy

In designing highway drainage, there are three major considerations; first, the safety of the traveling public, second, the design should be in accordance with sound engineering practices to economically protect and drain the highway, and third, in accordance with reasonable interpretation of the law, to protect private property from flooding, water soaking or other damage. In general, the hydraulic adequacy of structures is determined by the methods as outlined in this manual and performance records of structures in the same or similar locations.

8.1.3 Design Frequency

Federal and State governments have placed increasing emphasis on environmental protection over the last several years. Consequently the administrative rules established by regulatory agencies have made past practice of designing structures to accommodate flood frequencies of 25 and 50 years obsolete and unworkable. Thus, the design discharge for all bridges and box culverts covered under this chapter shall be the 100 year (Q_{100}) frequency flood. In floodplain management this is also referred to as the Regional or Base flood. Design frequency is determined from requirements in Federal Highway Administration (FHWA) directives and the co-operative agreement between Wisconsin Department of Transportation (DOT) and Wisconsin Department of Natural Resources (DNR). The following publications are suggested for guidance.

8.1.3.1 FHWA Directive

Title 23, Chapter 1, Sub Chapter G, Part 650, Subpart A of the FHWA – Federal-Aid Policy Guide, "*Location and Hydraulic Design of Encroachments on Flood Plains*", prescribes FHWA policy and procedures. Copies of this directive may be found on the FHWA website.

8.1.3.2 DNR-DOT Cooperative Agreement

The Wisconsin Department of Transportation and the Wisconsin Department of Natural Resources have signed a co-operative agreement to provide a reasonable and economical procedure for carrying out their respective duties in a manner that is in the total public interest. The provisions in this agreement establish the basic considerations for highway stream crossings. A copy of this agreement can be found in Figure 1, Procedure 20-30-1 of the Wisconsin DOT Facilities Development Manual (FDM).

8.1.3.3 DOT Facilities Development Manual

Refer to Procedures - Chapter 13 - Drainage Practice, Chapter 20 - Environmental Laws, Policies and Regulations and Chapter 21 - Environmental Documents, Reports and Permits.

8.1.4 Hydraulic Site Report

The "Stream Crossings Structure Survey Report" shall be submitted for all bridge and box culvert projects. When submitting preliminary structure plans for a stream crossing, a hydraulic site report shall also be included. A check list of the various discussion items that need to be provided in the hydraulic site report is included as 8.6 Appendix 8-A. Plan survey datum must conform to datum in use by local zoning authorities. In most cases elevations are referenced to the National Geodetic Vertical Datum (NGVD) of 1929, or to the North American Vertical Datum of 1988 (NAVD 88). The Hydraulic Site Report discusses and documents the hydrologic, hydraulic, site conditions, and all other pertinent factors that influence the type, size, and location of the proposed structure.

8.1.5 Hydraulic Design Criteria for Temporary Structures

The basic design criteria for temporary structures will to be the ability to pass a 5-year storm (Q5) with only 0.5 feet of backwater over existing conditions. This criteria is only a general guideline and site specific factors and engineering judgment may indicate that this criteria is inappropriate. Separate hydraulic design criteria should be used for the design of temporary construction causeways. Factors that should be considered in the design of temporary structures and approach embankments are:

- Effects on surrounding property and buildings
- Velocities that would cause excessive scour
- Damage or inconvenience due to failure of temporary structure
- DNR concerns

- Temporary roadway profile
- Structure depths will be 36” for short spans and 48” or more for longer spans.

If possible and practical, the temporary roadway profile should be designed and constructed in such a manner that infrequent flood events are not obstructed from overflowing the temporary profile and creating excessive backwaters upstream of the construction. The temporary roadway profile should provide adequate clearance for the temporary structure.

The roadway designer should indicate the need for a temporary structure on the Stream Crossing Structure Survey Report. Preliminary and Final plans should indicate the hydraulic parameters of the temporary structure. The required parameters are the 5-year flood discharge (Q5), the 5-year high-water elevation (HW5), and the flow area of the temporary structure required to pass the 5-year flood (Abr).

8.1.6 Erosion Control Parameters

In order to assist designers in determining the appropriate erosion control measures to be provided at Bridge construction site, preliminary and final plans should indicate the 2-year flood discharge (Q2), 2-year velocity, and the 2-year high-water elevation (HW2).

8.1.7 Bridge Rehabilitation and Hydraulic Studies

Generally no hydraulic study will be required in bridge rehabilitation projects that do not involve encroachment to the Base Floodplain. This includes entire super structure replacement provided that the substructure and berm configuration remain unchanged and the low cord elevation is not significantly lowered.

The designer should consider historical high-water elevations, Flood Insurance Studies and the potential of inundation when choosing the replacement superstructure type. The risk of damage to the structure as the result of Scour should also be considered.

8.2 Hydrologic Analysis

The first step in designing a hydraulic structure is to determine the design discharge for the waterway. The problem is particularly difficult for small watersheds, say under five square miles, because the smaller the area, the more sensitive it is to conditions which affect runoff and the less likely there are runoff records for the area.

Acceptable methods of determining the design discharge for the 100 year flood shall be based on the guidelines contained in the *State Administrative Code NR 116.07, Wisconsin's Floodplain Management Program*[1]. Generally, a minimum of two methods should be used in determining a design discharge.

The most frequently used methods for determining the design discharge for bridges and box culverts in the State of Wisconsin are discussed below.

8.2.1 Regional Regression Equations

The U. S. Geological Survey (USGS) in cooperation with the Wisconsin Department of Transportation prepared a report entitled *Flood Frequency Characteristics of Wisconsin Streams*[2] which considers the flood potentials for a site using regional regression equations based on flood data from gaging stations on Wisconsin's rivers and streams. The flood-frequency regression equations are correlated with three or more of seven parameters, namely, drainage area, main-channel slope, storage, forest cover, mean annual snowfall, precipitation intensity index, and soil permeability. These equations are applicable to all drainage areas in Wisconsin except for highly regulated streams, and highly urbanized areas of the state.

8.2.2 Watershed Comparison

The results obtained from the above regression equations should be compared to similar gaged watersheds listed in reference (2) above using the area transfer formulas and procedures detailed in that document. A good discussion and examples of the use of regression equations and basin comparison methods can be seen in the WisDOT Facilities Development Manual, Procedure 13-10-5. The flood frequency discharges listed in reference (2) are for flood records up to the year 2000. More years of data are available from the USGS for most of the gaged watersheds.

The flood frequency discharges for the gaged watersheds can be updated past water year 2000 by using the Log-Pearson Type III distribution method as described in *Bulletin #17B entitled Guidelines For Determining Flood Flow Frequency*[3] and the guidelines for weighting the station skew with the generalized skew in *NR116.07, Wisconsin's Floodplain Management Program*[1].

8.2.3 Flood Insurance and Floodplain Studies

The Federal Emergency Management Agency (FEMA) had contracted for detailed flood studies throughout Wisconsin. They were developed for floodplain management and flood insurance purposes. These Flood Insurance Studies (FIS) which are on file with Floodplain-

Shoreland Management Section of the Wisconsin Dept. of Natural Resources (DNR) contain discharge values for many sites. These studies, along with other various floodplain studies, may be obtained from the DNR's Floodplain Analysis Interactive Map by using the following link:

http://dnr.wi.gov/topic/floodplains/mapindex.html

8.2.4 Natural Resources Conservation Service

For small watersheds in urban and rural areas, the National Resources Conservation Service (NRCS) has developed procedures to calculate storm runoff volumes, peak rates of discharge, hydrographs and storage volumes. The procedure is documented in *Technical Release 55 Urban Hydrology for Small Watersheds*[4].

8.3 Hydraulic Design of Bridges

Bridge design for roadway stream crossings requires analysis of the hydraulic characteristics for both the "existing conditions" and the "proposed conditions" of the project site. A thorough hydraulic analysis is essential to providing a properly sized, safe and economical bridge design and assessing the relative impact that the proposed bridge has on the floodplain. The following subsections discuss design considerations and hydraulic design procedures for bridges. See 8.6 Appendix 8-A for a checklist of items that need to be considered and included in the Hydraulic/Sizing report for stream crossing structures.

8.3.1 Hydraulic Design Factors

Several hydraulic factors dictate the design of both the bridge and the approach roadway within the floodplain limits of the project site. The critical hydraulic factors for design consideration are:

8.3.1.1 Velocity

Velocity through the bridge opening is a major design factor. Velocity relates to the scour potential in the bridge opening and the development of scour areas adjacent to the bridge. Examination of the "existing conditions" model, existing site conditions, soil conditions, and flooding history will give good insight to acceptable design velocity. Generally, velocities through bridges of less than 10 feet per second are acceptable.

8.3.1.2 Roadway Overflow

The vertical alignment of the approach grade is a critical factor in the bridge design when roadway overflow is a design consideration. The two important design features of roadway overflow is overtopping velocity and overtopping frequency. See 8.3.2.6.2

8.3.1.3 Bridge Skew

When a roadway is at a skew angle to the stream or floodway, the bridge shall also be at a skew to the roadway with the abutments and piers parallel to the flow of the stream. The hydraulic section through the bridge shall be the skewed section normal to the flow of the stream. Generally, in the design of stream crossing, the skew of the structure should be varied in increments of 5 degrees where practical. Improper skew can greatly aggravate the magnitude of scour.

8.3.1.4 Backwater and High-water Elevation

Roadways and bridges are generally restrictions to the normal flow of floodwaters and increase the flood profile in most situations. The increase in the flood profile is referred to as the backwater and the resultant upstream water surface elevation is referred to as the High-Water Elevation (HW).

The high-water elevation or backwater calculations at the bridge are directly related to the bridge size and roadway alignment, which dictates all of the aforementioned hydraulic design factors. A significant design consideration when computing backwater is the potential for increasing flood damage for upstream property owners. The Cooperative Agreement between the Wis. Department of Natural Resources (DNR) and Wis. Department of Transportation (DOT) (see FDM procedure 20-30-1, Figure 1) defines the policy for high-water elevation design. That portion of the Cooperative Agreement relating to floodplain considerations is based on the Wisconsin Adm. Rule NR116, “Wisconsin Floodplain Management Program”. It is advisable to thoroughly study both documents as they can significantly influence the hydraulic design of the bridge.

One very subtle backwater criteria which is not addressed under the guidelines of the DNR-DOT Cooperative Agreement, is the backwater produced for flood events less than the 100 year frequency flood. Design consideration should be given to the more frequent flood events when there is potential for increasing the extent and frequency of flood damage upstream.

8.3.1.5 Freeboard

Freeboard is defined as the vertical distance between the low cord elevation of the bridge superstructure and the high-water elevation. A freeboard of 2.0 feet is the desirable minimum for all types of superstructures. However, economics, vertical and horizontal alignment, and the scope of the project may force a compromise to the 2 foot minimum freeboard. For these situations, close evaluation shall be made of the type and amount of debris and ice that would pass through the structure. Freeboard should be computed using the low chord elevation at the upstream face on the lower end of the bridge. The calculated 100-year high water elevation at a cross section that is approximately one bridge length upstream should be used to check freeboard.

It has become common practice that if debris and ice are a potential problem, or adequate freeboard cannot be provided, a concrete slab superstructure is preferred. A girder superstructure may be susceptible to damage when ice and/or debris is a significant problem. Girder structures are more susceptible to damage associated with buoyancy and lateral hydrostatic forces. In situations where the superstructure may be inundated during major flood events, it is recommended that the girders be anchored, tied or blocked so they cannot be pushed or lifted off the substructure units by hydraulic forces. In addition, air vents near the top of the girder webs can allow entrapped air to escape and thus may reduce buoyancy forces. The use of Precast Pretensioned Slab and Box Sections is allowed where desirable freeboard cannot be provided and conventional cast in place slabs cannot be employed. The following requirements should be met:

- Precast Pretensioned Slab and Box Sections may be in the water for the 100-year flood. The designer will be responsible for ensuring the stability of the structure for buoyant and lateral forces.

- If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 5-year event, the Precast Pretensioned Slab and Box Sections must be cast solid.

- If Precast Pretensioned Slab and Box Sections are in contact with water for flood events equal to or less than a 100-year event, the void in Precast Pretensioned Slab and Box Sections must be cast with a non-water absorbing material.

8.3.1.6 Scour

Investigation of the potential for scour at the bridge site is a design consideration for the bridge opening geometry and size, as well as pier and abutment design. Bridges shall be designed to withstand the effects of scour from a super-flood (a flood exceeding the 100-year flood) without failing; i.e., experiencing foundation movement of a magnitude that requires corrective action. See 8.3.2.7. Generally, scour associated with a 100-year event without significant reduction in foundation factor of safety will accomplish this objective. For situations where a combination of flow through a bridge and over the roadway exist, scour should also be evaluated for flow conditions at the onset of flow over topping when velocity through the bridge may be the greatest.

8.3.2 Design Procedures

8.3.2.1 Determine Design Discharge

See 8.2 for procedures.

8.3.2.2 Determine Hydraulic Stream Slope

The primary method of determining the hydraulic slope of a stream is surveying the water surface elevation through a reach of stream 1500 feet upstream to 1500 feet downstream of the site. Intermediate points through this reach should also be surveyed to detect any significant slope variation.

There are situations, particularly on flat stream profiles, where it is difficult to determine a realistic slope using survey data. This will occur at normal water surface elevation at the mouth of a stream, upstream of a dam, or other significant restriction in the stream. In this case a USGS 7-1/2" quadrangle map and existing flood studies of the stream can be investigated to determine a reasonable stream slope.

8.3.2.3 Select Floodplain Cross-Section(s)

Generally, a minimum of two floodplain valley cross-section(s) are required to perform the hydraulic analysis of a bridge. The sections shall be normal to the stream flow at flood stage and approximately one bridge length upstream and downstream of the structure. A detailed cross-section of one or both faces of the bridge will also be required. If the section is skewed to the flow, the horizontal stationing shall be adjusted using the cosine of the skew angle.

If the downstream boundary condition of the hydraulic model is using normal depth, then the most downstream cross-section in the model should be located far enough downstream from the bridge and should reflect the natural floodplain conditions.

Field survey cross-sections will be needed when a contour map is plotted using stereographic methods. A field survey section is needed for that portion below the normal water surface.

Cross-sections taken from contour maps are acceptable when the information is supplemented with field survey sections and data. Additional sections may be required to develop a proper hydraulic model for the site.

The hydraulic cross-sections should not include slack water portions of the flood plain or portions not contributing to the downstream movement of water.

Refer to FDM Procedure 9-55-5 for a discussion of Drainage Structure Surveys.

8.3.2.4 Assign “Manning n” Values to Section(s)

“Manning n” values are assigned to the cross-section sub-areas. Generally, the main channel will have different “manning n” values than the overbank areas. Values are chosen by on-site inspection, pictures taken at the section, and use of aerial photos defining the extent of each “n” value. There are several published sources on open channel hydraulics which contain tables for selecting appropriate “n” values. See 8.5 References (5) and (6).

8.3.2.5 Select Hydraulic Model Methodology

There are several public and private computer software programs available for modeling open channel hydraulics, bridge hydraulics, and culvert hydraulics. Three public domain computer software programs that are most prevalent and preferred in Wisconsin bridge design work are “HEC-RAS”, “WSPRO” and “HY8”.

The HEC-RAS program is currently the most widely used methodology for floodplain and bridge hydraulic modeling. HEC-RAS has more options and capabilities than WSPRO when modeling complex floodplains and requires a greater amount of expertise to apply. HEC-RAS should be used where existing HEC-2 data is available from a previous Flood Insurance Study. The WSPRO methodology is tailored specifically for bridge hydraulics with many appropriate default coefficients and analysis options. More information on these two programs is given below. “HY8” is a FHWA sponsored culvert analysis package based on the FHWA publication “Hydraulic Design of Highway culverts” (HDS-5), see 8.5 Reference (13).

1. HEC-RAS

   The hydrologic Engineering Center’s River Analysis System (HEC-RAS) is the first of the U.S. Army Corps of Engineers “Next Generation” software packages. It is the successor to the HEC-2 program, which was originally developed by the Corps of Engineers in the early 1970’s. HEC-RAS includes several data entry, graphing, and reporting capabilities. It is well suited for modeling water flowing through a system of open channels and computing water surface profiles to be used for floodplain management and evaluation of floodway encroachments. HEC-RAS can also be used for bridge and culvert design and analysis and channel modification studies.

   For a complete treatise on the methodology of the program, see 8.5 reference (7), (8) and (9). The HEC-RAS program and supporting documentation can be downloaded from the U.S. Army Corps of Engineers web site: http://www.hec.usace.army.mil/software/hec-ras/. A list of vendors for HEC-RAS is also available on this web site.

2. WSPRO

   "Water Surface Profiles (WSPRO)" is a computer program developed by the U.S. Geological Survey under contract with Federal Highway Administration. WSPRO was specifically oriented toward hydraulic design of highway bridges although it is equally suitable for water surface profile computations unrelated to highway and bridge design.

   The program uses bridge backwater computations based on analyses documented in the USGS publication entitled Measurement of Peak Discharge at Width Contractions by Indirect Methods, see 8.5 reference (10).

   For a complete treatise on the methodology of the program, see 8.5 reference (11) and (12). The WSPRO program and supporting documentation can be downloaded from the following FHWA web site, or can be obtained through "McTrans" or "PcTrans". See 8.7 Appendix 8-B.

   http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm

3. HY8

   HY8 is a computer program that uses the FHWA culvert hydraulic approaches and protocols as documented in the publication "Hydraulic Design Series 5: Hydraulic Design of Highway Culverts" (HDS-5). See 8.5 reference (13). HY8 can perform hydraulic computations for circular, rectangular, elliptical, metal box, high and low profile arch, as well as user defined geometry culverts. FHWA recently released a new Windows based version of the HY-8 culvert program. The methodology used by HY8 is discussed in 8.4.2.4. This program can be downloaded from the FHWA web site: http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm.

8.3.2.6 Develop Hydraulic Model

First, a hydraulic model shall be developed for the "existing conditions" at the bridge site. This shall become the basis for hydraulic design of "proposed conditions" for the project and allows for an assessment of the relative hydraulic changes associated with the proposed structure. Special attention should be given to historic high-water and flood history, evidence of scour (high velocity), roadway overtopping, existing high-water, and compatibility with existing Flood Insurance Study (FIS) profiles. When current information and/or estimates of site conditions or flows differ significantly from adopted regulatory information (FIS), it may be necessary to compute both "design" and "regulatory" existing and proposed conditions.

There are a number of encompassing features of a steady state (flow is constant) hydraulic model for a roadway stream crossing. They include the natural adjacent floodplain, subject structure, any supplemental structures, and the roadway. Accurate modeling and calculations need to account for all potential conveyance mechanisms. Generally, most modern step-backwater methodologies can incorporate all of the above elements in the evaluation of hydraulic characteristics of the project site.

8.3.2.6.1 Bridge Hydraulics

The three most common types of flow through bridges are free surface flow (low flow), free surface (unsubmerged) orifice flow and submerged orifice flow. The latter two are also referred to as pressure flow. All of the above flow conditions may also occur simultaneously with flow over the roadway.

There are situations in which steep stream slopes are encountered and the flow may be supercritical (Froude No. > 1). This is a situation in which theoretically no backwater is created. For critical and supercritical flow situations the profile calculation would proceed from upstream to downstream. If this situation is encountered, the accuracy of the hydraulic model may be suspect and it is questionable whether the bridge should impose any constrictions on the stream channel. Sufficient clearance should be provided to insure that the superstructure will not come in contact with the flow.

Generally, in Wisconsin, most natural stream flow is in a sub-critical (Froude No. < 1) regime. Therefore, the water surface profile calculation will proceed from downstream to upstream.

Sample bridge hydraulic problems using HEC-RAS can be found in the HEC-RAS Applications Guide[9].

8.3.2.6.2 Roadway Overflow

One potential element in developing a hydraulic model for a stream crossing is roadway overflow. It is sometimes necessary to compute flow over highway embankments in combination with flow through structure openings. Most automated methodologies will incorporate the division of flow through a structure and over the road in determination of the solution. The WSPRO methodology will conduct the “combined flow” solution and internally determine and adjust the coefficient of discharge for both the structure and roadway weir section. Other methodologies, such as HEC-RAS, rely on user defined coefficients for both the structure and roadway flow solutions. The discharge equation and coefficients for flow over a highway embankment are given in this section.

The geometry and flow pattern for a highway embankment are illustrated in Figure 8.3-4. Under free flow conditions critical depths occur near the crown line. The head (H) is referred to the elevation of the water above the crown, and the length (L), in direction of flow, is the distance between the points of the upstream and downstream embankment faces (edge of shoulder). The length (B) of the embankment has no influence on the discharge coefficient.

The weir discharge equation is:

$$Q = k_t \cdot C_f \cdot B \cdot H^{3/2}$$

Where:

Q	=	discharge
C_f	=	coefficient of discharge for free flow conditions
B	=	length of flow section along the road normal to the direction of flow
H	=	total head = h + h_V
k_t	=	submergence factor

The length of overflow section (B) will be a function of the roadway profile grade line and depth of over-topping (h). Coefficient (C_f) is obtained by computing h/L and using Figure 8.3-1 or Figure 8.3-2, for paved or gravel roads.

The degree of submergence of a highway embankment is defined by ration ht/H. The effect of submergence on the discharge coefficient (C_f) is expressed by the factor k_t as shown in Figure 8.3-3. The factor k_t is multiplied by the discharge coefficient (C_f) for free-flow conditions to obtain the discharge coefficient for submerged conditions. For roadway overflow conditions with high degree of submergence, HEC-RAS switches to energy based calculations of the upstream water surface. The default maximum submergence is 0.95, however that criterion may be modified by the user.

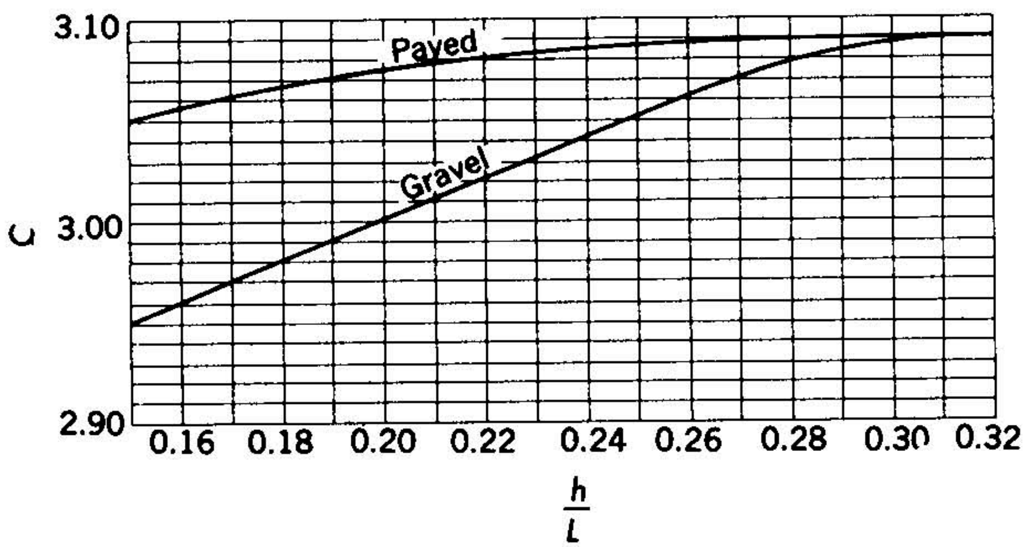


Figure 8.3-1
Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios > 0.15

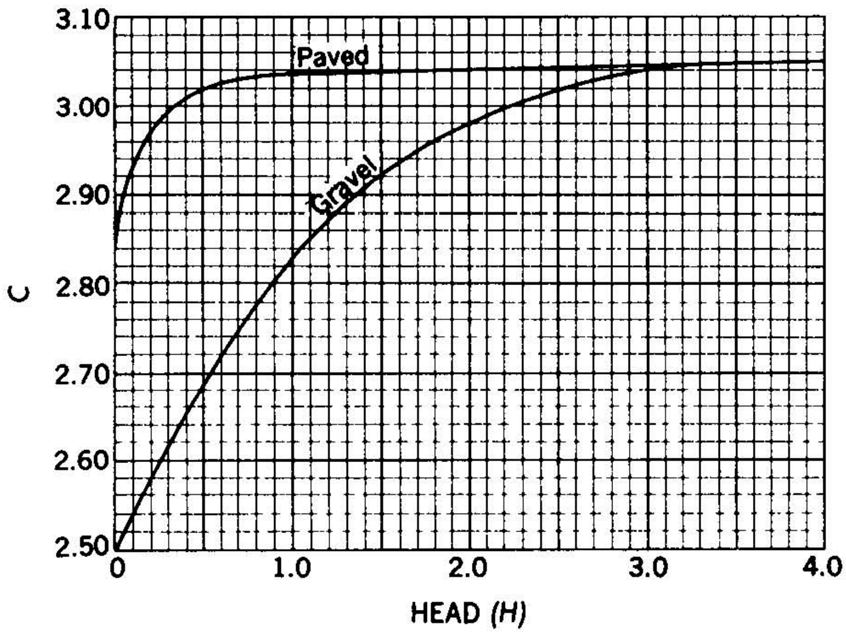


Figure 8.3-2

Discharge Coefficients, C_f, for Highway Embankments for H/L Ratios < 0.15

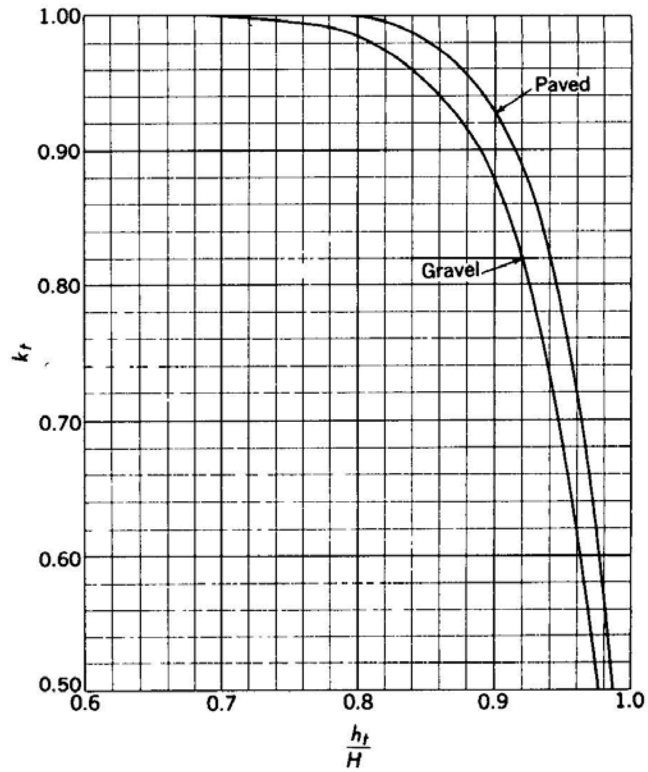


Figure 8.3-3

Definition of Adjustment Factor, k_t, for Submerged Highway Embankments

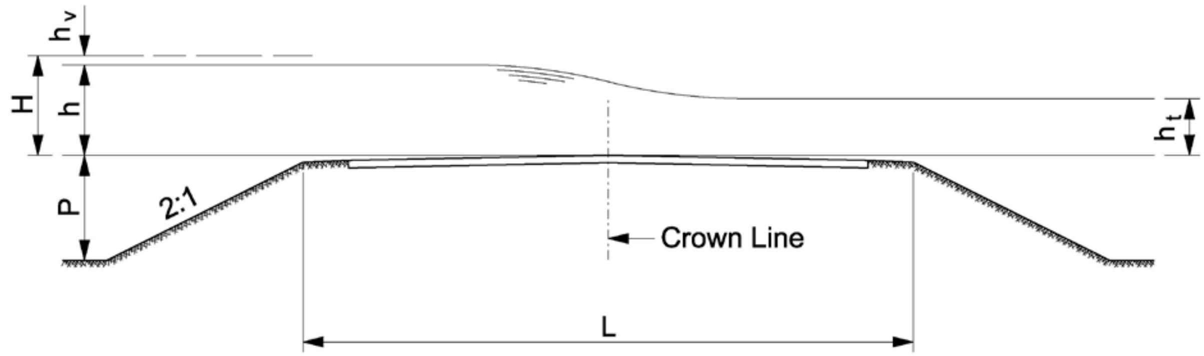


Figure 8.3-4
Definition Sketch of Flow Over Highway Embankment

8.3.2.7 Conduct Scour Evaluation

Evaluating scour potential at bridges is based on recommendations and background from FHWA Technical Advisory "*Evaluating Scour at Bridges*" dated October 28, 1991 and procedures from the *FHWA Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges, Fourth Edition*, May 2001[14], and *Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures, Third Edition*, March 2001[15]. Consult FHWA's website for the most current versions of the above publications.

All bridges shall be evaluated to determine the vulnerability to scour. In the FHWA publication *Recording and Coding Guide for Structure Inventory and Appraisal of the Nation's Bridges*[16], a code system has been established for evaluation. A section in this guide "Item 113 - Scour Critical Bridges" uses a single-digit code to identify the status of the bridge regarding its vulnerability to scour. The most current version of the Item 113 Scour Coding Guide can be found here: https://www.fhwa.dot.gov/engineering/hydraulics/policymemo/revguide.cfm.

There are three main components of total scour at a bridge site. They are Long-term Aggradation and Degradation, Contraction Scour, and Local Scour. In addition, lateral migration of the stream must be assessed when evaluating total scour at substructure units. Contraction and local scour will be evaluated in the context of clear-water and live bed scour conditions. In most of the methods for determining individual scour components, hydraulic characteristics at the approach section are required. The approach section should be understood as the cross section located approximately one bridge length upstream of the bridge opening.

8.3.2.7.1 Live Bed and Clear Water Scour

Clear-water scour occurs when there is insignificant or no movement (transport) of the bed material by the flow upstream of the crossing, but the acceleration of flow and vortices created by the piers or abutments causes the bed material in the vicinity of the crossing to move.

Live-bed scour occurs when there is significant transport of bed material from the upstream reach into the crossing.

8.3.2.7.2 Long-term Aggradation and Degradation

Aggradation is the deposition of eroded material in the stream from the upstream watershed. Degradation is the scouring (removal) of the streambed resulting from a deficient supply of sediment. These are subtle long term streambed elevation changes. These processes are natural in most cases. However, unnatural changes like dam construction or removal, as well as urbanization may cause Aggradation and Degradation. Excellent reference on this subject and the geomorphology of streams is the FHWA publication *Highways in the River Environment*[17], *HEC-18, Evaluating Scour at Bridges*[14], and *HEC-20, Stream Stability at Highway Structures*[15].

8.3.2.7.3 Contraction Scour

Generally, Contraction scour is caused by bridge approaches encroaching onto the floodplain and decreasing the flow area resulting in an increase in velocity through a bridge opening. The higher velocities are able to transport sediment out of the contracted area until an equilibrium is reached. Contraction scour can also be caused by short term changes in the downstream water surface elevation, such as bridges located on a meander bend or bridges located in the backwater of dams with highly fluctuating water levels. See 8.5 reference (14) & (15) for discussion and methods of analysis. If a pressure flow condition exists at the bridge opening, then vertical contraction scour must be evaluated. Reference HEC-18 for a description of the method used to estimate this scour component.

Computing Contraction Scour.

1. Live-Bed Contraction Scour

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1}$$

Where:

y_s	=	y_2-y_0 = Average scour depth, ft
y_1	=	Average depth in the upstream main Channel, ft
y_2	=	Average depth in the contracted section, ft

y_0 = Existing depth in the contracted section before scour, ft

Q_1 = Flow in upstream channel transporting sediment, ft^3/s

Q_2 = Flow in contracted channel, ft^3/s

W_1 = Bottom Width of upstream main channel, ft

W_2 = Net bottom Width of channel at contracted section, ft

k_1 = Exponent for mode of bed material transport, 0.59-0.69 see 8.5 ref. (14)

2. Clear-Water Contraction Scour

$$y_2 = \left[\frac{Q^2}{130 \cdot D_m^{\frac{3}{2}} \cdot W^2} \right]^{\frac{3}{7}}$$

Where:

y_s = y_2-y_0 = Average scour depth, ft

y_2 = Average depth in the contracted section, ft

y_0 = Existing depth in the contracted section before scouring, ft

Q = Discharge through the bridge associated with W, ft^3/s

D_m = Diameter of the smallest nontransportable particle ($1.25D_{50}$), ft

D_{50} = Median Diameter of the bed material (50% smaller than), ft

W = Net bottom Width of channel at contracted section, ft

8.3.2.7.4 Local Scour

Local scour is the removal of material from around a pier abutment, spur dike, or the embankment. It is caused by an acceleration of the flow and/or resulting vortices induced by obstructions to flow.

1. Pier Scour & Colorado State University's (CSU) Equation

   The recommended equation for determination of pier scour is the CSU's equation. Velocity is a factor in calculating the Froude Number. Therefore it is applicable where a hydraulic model of the bridge is available. The equation and appropriate charts and tables are shown below in Table 8.3-1, Table 8.3-2, Table 8.3-3 and Figure 8.3-5. See 8.5 reference (14) for a complete discussion of the CSU Equation.

The CSU equation for pier scour is:

$$\frac{y_s}{a} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot \left(\frac{y_1}{a}\right)^{0.35} \cdot Fr_1^{\ 0.43}$$

Where:

y_s	=	Scour depth, ft
y_1	=	Flow depth directly upstream of the pier, ft
A	=	Pier width, ft
Fr_1	=	Froude number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1	=	Mean Velocity of flow directly upstream of the pier, ft/s
g	=	Acceleration of gravity, 32.2 ft/s^2
K_1	=	Correction Factor for pier nose shape (see Table 8.3-1 and Figure 8.3-5)
K_2	=	Correction Factor for angle of attack of flow (see Table 8.3-2)
K_3	=	Correction Factor for bed condition (see Table 8.3-3)
K_4	=	Correction Factor for armoring by bed material 0.7 - 1.0 (see 8.5 reference 14)

Correction Factor, K_1, for Pier Nose Shape (HEC-18 Table 2)	
Shape of Pier Nose	K_1
(a) Square Nose	1.1
(b) Round Nose	1.0
(c) Circular Cylinder	1.0
(d) Group of Cylinders	1.0
(e) Sharp Nose	0.9

Table 8.3-1
Correction Factor, K_1, for Pier Nose Shape

Correction Factor, K_2, for Angle of Attack, Θ, of the Flow (HEC-18 Table 3)			
Angle	L/a = 4	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, ft a = pier width, ft			

Table 8.3-2
Correction Factor, K_2, for Angle of Attack, θ, of the Flow

Increase in Equilibrium Pier Scour Depths, K_3, for Bed Conditions (HEC-18 Table 4)		
Bed Condition	Dune Height, ft	K_3
Clear – water Scour	N/A	1.1
Plane Bed and Antidune Flow	N/A	1.1
Small Dunes	$3 > H \geq 0.6$	1.1
Medium Dunes	$9 > H \geq 3$	1.2 to 1.1
Large Dunes	$H \geq 9$	1.3

Table 8.3-3
Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition

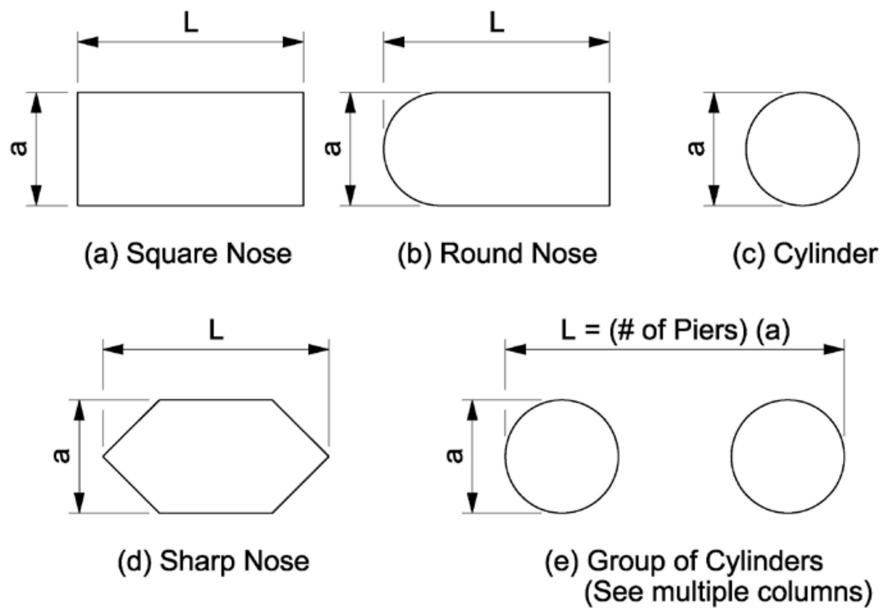


Figure 8.3-5
Common Pier Shapes

2. Abutment Scour Equations

Abutment scour analysis is dependent on equations that relate the degree of projection of encroachment (embankment) into the flood plain. Several equations were developed to estimate abutment scour depths, however lack of field data to verify any one equation causes doubt on the reliability of these scour estimates. This is one of the reasons heavy riprap underlain with geotextile fabric used to resist scour as described in the construction specifications at most stream crossing abutments.

Three methods are presented in the FHWA publication HEC-18 "Evaluating Scour at Bridges". The HIRE equation can be used when L/y1 is greater than 25, where L is the length of embankment projected and normal to the flow (ft), and y_1 is depth of flow at the abutment on the overbank or in the main channel (ft). For lower values of L/y1, the live-bed Froehlich equation can be used, which incorporates the effective embankment length. The user needs to refer to the publication HEC-18, see 8.5 reference (14), for a discussion of the applicability of the equations presented and further definitions of the parameters used in these equations. In addition, common hydraulic modeling programs used for bridge design, such HEC-RAS and WSPRO, include routines to calculate abutment scour. Designers are cautioned to closely examine how the parameters that are used in these automated routines are defined. The third approach presented in HEC-18 was recently developed under NCHRP Project 24-20. This method includes equations that encompass a range of abutment types and locations, as well as flow conditions. The primary advantage of this approach is that the equations are more physically representative of the abutment scour process, but it also avoids using the effective embankment length, which can be difficult to determine accurately. This approach computes total scour, rather than just local scour, at the abutment. Reference HEC-18 for a detailed description of the NCHRP approach and equations.

Froelich's Live-Bed Scour at Abutments

$$\frac{y_s}{y_a} = 2.27 \cdot K_1 \cdot K_2 \cdot \left(\frac{L'}{y_a}\right)^{0.43} \cdot F_r^{\ 0.61} + 1$$

y_s = Scour depth, ft

y_a = Average depth of flow on the floodplain, ft

L' = Length of active flow obstructed by the embankment, ft

K_1 = Coefficient for abutment shape (see Table 8.3-4)

K_2 = Coefficient for angle of embankment to flow (see 8.5 reference 14)

Fr = Froude number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$

Where:

V_e = Q_e/A_e, ft/s

g = Acceleration of gravity, 32.2 ft/s^2

A_e = Flow Area of approach cross section obstructed by embankment, ft^2

Q_e = Flow obstructed by abutment and approach embankment, ft^3/s

The HIRE Equation for Live-Bed Scour at Abutments

$$\frac{y_S}{y_1} = 4 \cdot Fr_1^{\ 0.33} \cdot \frac{K_1 \cdot K_2}{0.55}$$

y_s = Scour depth, ft

y_1 = Depth of flow at the abutment on the overbank or in the main channel, ft

K_1 = Coefficient for abutment shape (see Table 8.3-4)

K_2 = Coefficient for skew angle of abutment to flow (see 8.5 reference 14)

Fr_1 = Froude number based on velocity and depth adjacent to and upstream of the abutment

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

Table 8.3-4
Abutment Shape Coefficients (K_1)

The Froehlich and HIRE equations often predict excessively conservative abutment scour depths. This is due to the fact that these equations were developed based on results of experiments in laboratory flumes and did not reflect the typical geometry or flow distribution associated with roadway encroachments on floodplains. However, since the NCHRP equations are more physically representative of the abutment scour process, greater confidence can be placed in the scour depths resulting from this approach.

8.3.2.7.5 Design Considerations for Scour

Provide adequate free board (2 feet desirable) to prevent occurrences of pressure flow conditions.

Pier foundation elevations on floodplains should be designed considering the potential of channel or thalweg migration over the design life of the structure.

Align all substructure units and especially piers with the direction of flow. Improper alignment may significantly increase the magnitude of scour.

Piers in the water should have a rounded or streamline nose to reduce turbulence and related scour potential.

Spill-through (sloping) abutments are less vulnerable to scour than vertical wall abutments.

The Froehlich and HIRE equations used to estimate the magnitude of abutment scour were developed in a laboratory under ideal conditions and lack adequate field verification. These equations may tend to over estimate the magnitude of scour. These equations should be incorporated with a great deal of discretion.

8.3.2.8 Select Bridge Design Alternatives

In most design situations, the “proposed bridge” design will be based on the various pertinent design factors discussed in 8.3.1. They will dictate the final selection of bridge length, abutment design, superstructure design and approach roadway design. The Hydraulic/Site report should adequately document the site characteristics, hydrologic and hydraulic calculations, as well as the bridge type and size alternatives considered. See 8.6 Appendix 8-A for a sample check list of items that need to be included in the Hydraulic/Site Report.

8.4 Hydraulic Design of Box Culverts

Box culverts are an efficient and economical design alternative for roadway stream crossings with design discharges in the 300 to 1500 cfs range. As a general guide culvert pipes are best suited for smaller discharge values while bridges are better suited for larger values. Although multi-cell box culverts are designed for larger discharges, the larger size culverts tend to lose the hydraulic and economic advantage over bridges. The following subsections discuss the design considerations and hydraulic design procedures for box culverts.

8.4.1 Hydraulic Design Factors

As in the hydraulic design of bridges, several hydraulic factors dictate the design of both the culvert and approach roadway. The critical hydraulic factors for design considerations are:

8.4.1.1 Economics

The best economics for box culvert design are realized with the culvert flowing full and producing a reasonable headwater depth (HW) within the boundary of other hydraulic and roadway design constraints.

For long box culverts, particularly on steep slopes, considerable savings can be realized by incorporating an improved inlet design known as "Tapered Inlets". The improved efficiency of the inlet where the inlet controls the headwater, will allow for design of a smaller culvert barrel. See 8.5 reference (13) for discussion on "Tapered Inlets".

8.4.1.2 Minimum Size

If the highway grade permits, a minimum five foot box culvert height is desirable for clean-out purposes.

8.4.1.3 Allowable Velocities and Outlet Scour

Generally, for velocities under 10 fps no riprap is needed at the discharge end of a box culvert, although close examination of local soil conditions is advisable.

For outlet velocities from 10-14 fps heavy riprap shall be used extending 15 to 35 feet from the end of the culvert apron.

For velocities greater than 14 fps energy dissipators should be considered. These are the most expensive means of end protection. See 8.4.2.7 for the hydraulic design of energy dissipators.

When heavy riprap is used it is carried up the slopes around the ends of the outlet apron to an elevation at mid-length of apron wing.

8.4.1.4 Roadway Overflow

See 8.3.1.2.

8.4.1.5 Culvert Skew

See 8.3.1.3.

8.4.1.6 Backwater and Highwater Elevations

The “Highwater elevation” commonly referred to as headwater for culverts, is the backwater created at the upstream end of the culvert. Although culverts are more hydraulically efficient and economical when flowing under a reasonable headwater, several factors shall be considered in determining an allowable highwater elevation. For further discussion see Section 8.3.1.4.

8.4.1.7 Debris Protection

Debris protection is provided where physical study of the drainage area indicates considerable debris collection. Where used, structural design of debris protection features should be part of the culvert design. The box culvert survey report must justify the need for protection. Sample debris protection devices are presented in the FHWA publication, *Hydraulic Engineering Circular No. 9, Debris Control Structures, Evaluation and Countermeasures.* See 8.5 reference (18).

8.4.1.8 Anti-Seepage Collar

Anti-seepage collars are used to prevent the movement of water along the outside of the culvert and the failure by piping of the fill next to the culvert. They are used in sandy fills where the culvert has a large headwater.

Collars are located at the midpoint and upstream quarter point on long box culverts. If only one collar is used, it is located far enough from the inlet to intercept the phreatic (zero pressure) line to prevent seepage over the top of the collar. See 8.5 reference (19).

A typical collar is shown in Figure 8.4-1 and is applicable to all single and twin box structures.

An alternate method of preventing seepage is to use a minimum one foot thick impervious soil blanket around the culvert inlet extending five feet over undisturbed embankment. The same effect can be obtained by designing seepage protection into the endwalls.

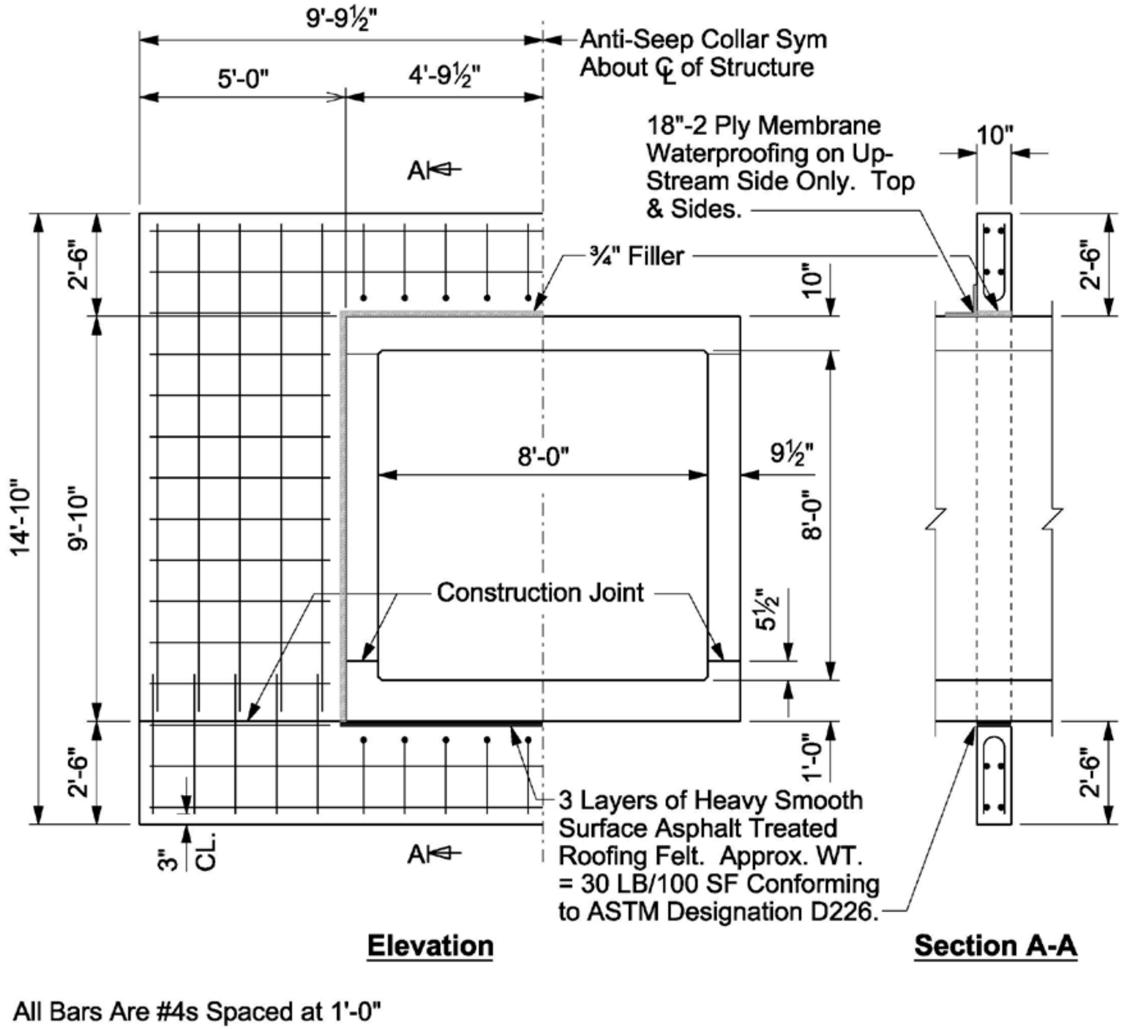


Figure 8.4-1
Anti-Seepage Collar

8.4.1.9 Weep Holes

The need for weep holes should be investigated for clay type soils with high fills, and should be eliminated in other cases.

If weep holes are necessary, alternate layers of fine and coarse aggregate are placed around the holes starting with coarse aggregate next to the hole.

8.4.2 Design Procedure

8.4.2.1 Determine Design Discharge

See 8.2 for procedures.

8.4.2.2 Determine Hydraulic Stream Slope

See 8.3.2.2 for procedures.

8.4.2.3 Determine Tailwater Elevation

The tailwater elevation is the depth of water in the natural channel computed at the outlet of the culvert. In situations of steeper slopes and small culverts, the tailwater is not a critical design factor. However, for mild slopes and larger culverts, the tailwater is a critical design factor. It may control the outlet velocity and depth of flow in the culvert.

The tailwater elevation is calculated using a typical section downstream of the outlet and performing a “normal depth” analysis. Most hydraulic engineering textbooks and handbooks include discussion of methods to calculate “normal depth” for symmetrical and irregular cross-sections in an open channel.

8.4.2.4 Design Methodology

The most prevalent design methodology for culverts is the procedure in the FHWA publication DHS No. 5, see 8.5 reference (13). It is highly recommended the designer first thoroughly study the methodologies presented in that publication.

Several computer software programs are available from public and private sources which use the same technique and methodology presented in HDS No. 5. One public domain computer program developed by FHWA entitled “HY8” is based on the HDS No. 5 manual. This program and documentation are available from the FHWA web site (see 8.7 Appendix 8-B). HEC-RAS and WSPRO also have culvert options using the same methodology. These programs have the capability of allowing the user to calculate the tailwater based on a downstream section and to calculate a combination of culvert and roadway overflow.

8.4.2.5 Develop Hydraulic Model

There are two major types of culvert flow: (1) flow with inlet control, and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area, and the inlet geometry at the entrance are of primary importance. Outlet control involves the consideration of the tailwater in the outlet channel, the culvert slope, the culvert roughness, and the length of the culvert barrel, as well as inlet geometry and cross-sectional area.

Another design of Inlet control which is used frequently is “Tapered Inlets” or improved inlets. The slope-tapered and side-tapered inlets are more efficient hydraulically, and can be a more economical design for long culverts in flow with inlet control.

In all culvert design, headwater depth (HW) or depth of water at the entrance to a culvert is an important factor in culvert capacity. The headwater depth is the vertical height from the culvert invert elevation at the entrance to the total energy elevation of the headwater pool (depth plus velocity head). Because of the low velocities at the entrance in most cases and difficulty in determining the velocity head for all flows, the water surface elevation and the total energy elevation at the entrance are assumed to be coincident.

The box culvert charts presented here are inlet and outlet control nomographs Figure 8.4-3 and Figure 8.4-4, and a critical depth chart Figure 8.4-6. Note the “Inlet Type” over the HW/D scales on Figure 8.4-3 and entrance loss coefficients “Ke” for inlet types on Figure 8.4-4. The following illustrative problems are examples of their use. Forms similar to Figure 8.4-2 are used for computation.

1. Outlet Control Problem.

   The information necessary to solve this problem is given in Figure 8.4-2.

   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D=1.08 from Figure 8.4-3.

   The HW = 1.08 (5 ft) = 5.4 ft.

   Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 180 ft. and type “C” inlet; H = 1.5 ft. from Figure 8.4-4, TW = 5.2 ft. = ho

   Then HW = H + h_o - LS_o = 1.5 ft. + 5.2 ft. - .2 ft. = 6.5 ft.

   Design HW is 6.5 ft. (outlet controls) and the outlet velocity is 7.2 f.p.s. No heavy riprap is needed at the discharge apron.

2. Inlet Control Problem.

   The information necessary to solve this problem is given in Figure 8.4-5.

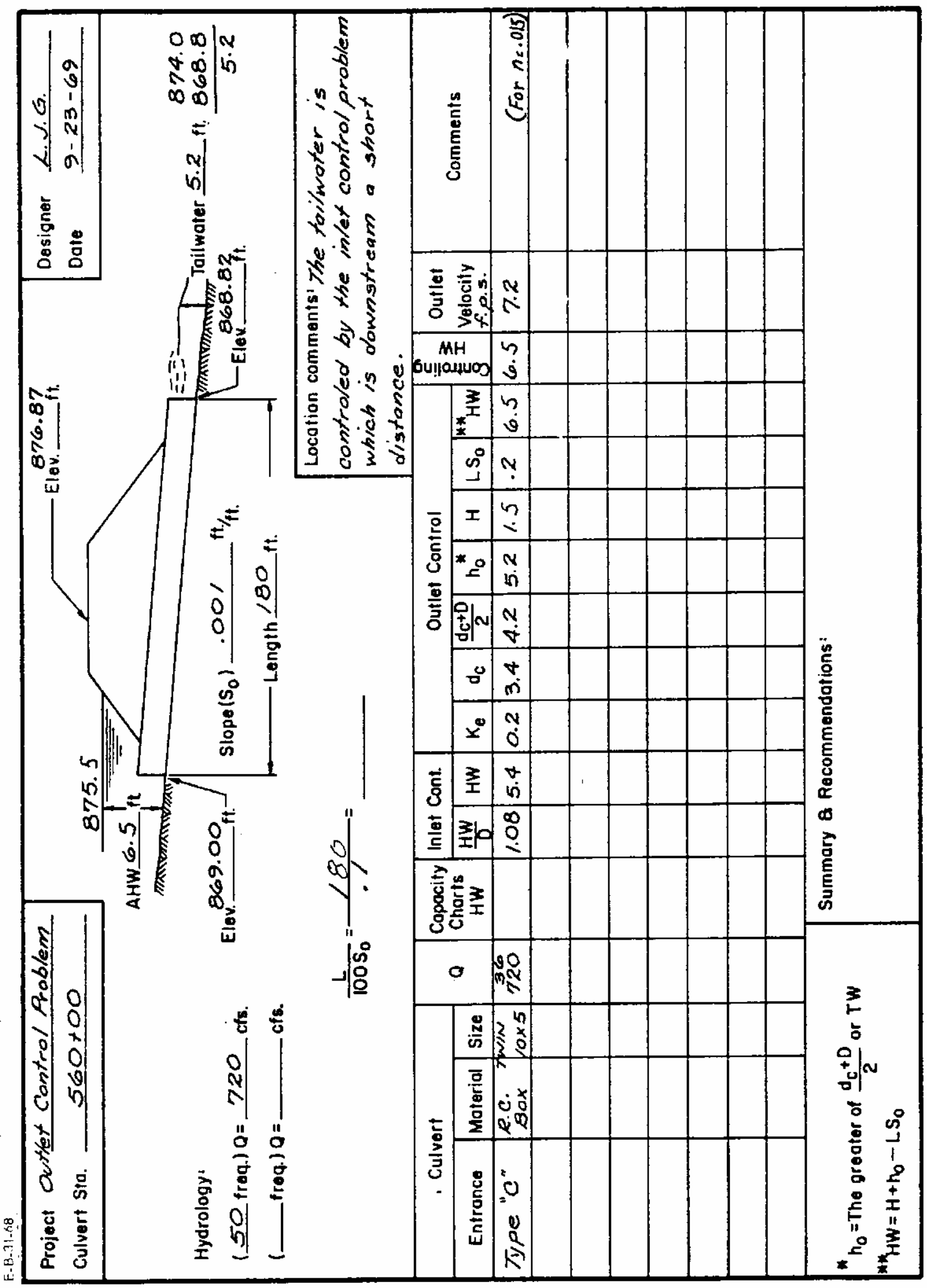
   Check for Inlet Control: For a Q/B value of 36 and a twin 10 x 5 box with type “C” inlet; HW/D = 1.08 from Figure 8.4-3.

   Then HW = 1.08 (5 ft.) = 5.4 ft.

   Check for Outlet Control: For Q = 720/2 = 360 cfs. Length = 132 ft. and type “C” inlet; H = 1.3 ft. from Figure 8.4-4. From Figure 8.4-6 critical depth = 3.4 ft. ho = (3.4 ft. + 5 ft.)/2 = 4.2 ft.

   Then HW = H + h_o - LS_o = 1.3 ft. + 4.2 ft. - .7 ft. = 4.8 ft.

   Design HW = 5.4 ft. (inlet control) and the outlet velocity is 11.0 f.p.s. Heavy riprap is needed at the discharge apron.



HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
E-B-31-68

Project *Outlet Control Problem*
Culvert Sta. *560+00*

Designer *L.J.G.*
Date *9-23-69*

Hydrology:
(*50* freq.) Q = *720* cfs.
(____ freq.) Q = ____ cfs.

$\frac{L}{100 S_0} = \frac{180}{.1} =$ ____

Location comments: *The tailwater is controled by the inlet control problem which is downstream a short distance.*

Culvert			Q	Capacity Charts HW	Inlet Cont.		Outlet Control							Controling HW	Outlet Velocity f.p.s.	Comments
Entrance	Material	Size			$\frac{HW}{D}$	HW	K_e	d_c	$\frac{d_c+D}{2}$	h_o*	H	LS_o	**HW			
Type "C"	R.C. Box	Twin 10x5	36 720		1.08	5.4	0.2	3.4	4.2	5.2	1.5	.2	6.5	6.5	7.2	(For n=.015)

* h_o = The greater of $\frac{d_c+D}{2}$ or TW

** $HW = H + h_o - LS_o$

Summary & Recommendations:

Figure 8.4-2
Culvert Computation Form

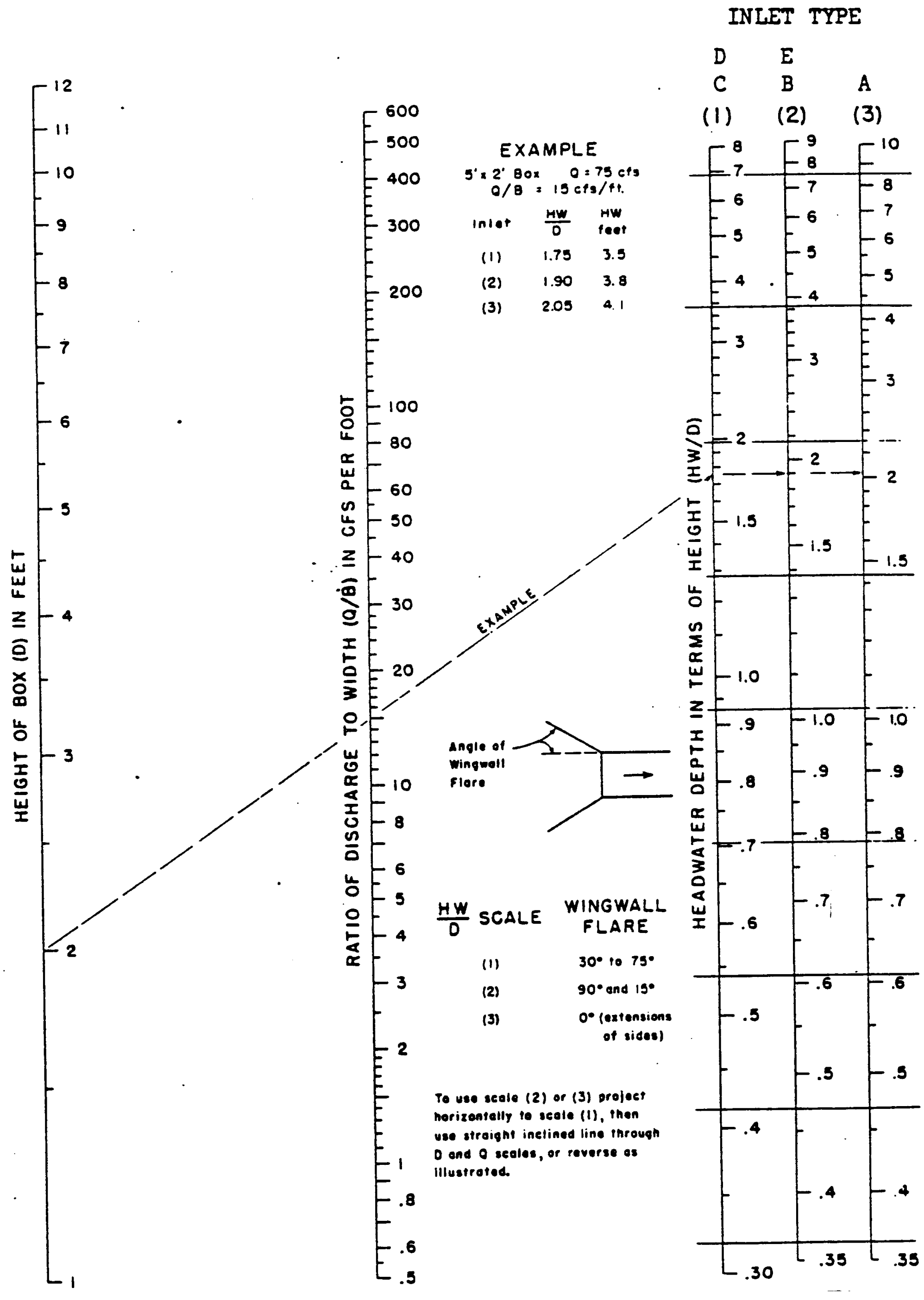


Figure 8.4-3
Headwater Depth for Box Culverts with Inlet Control

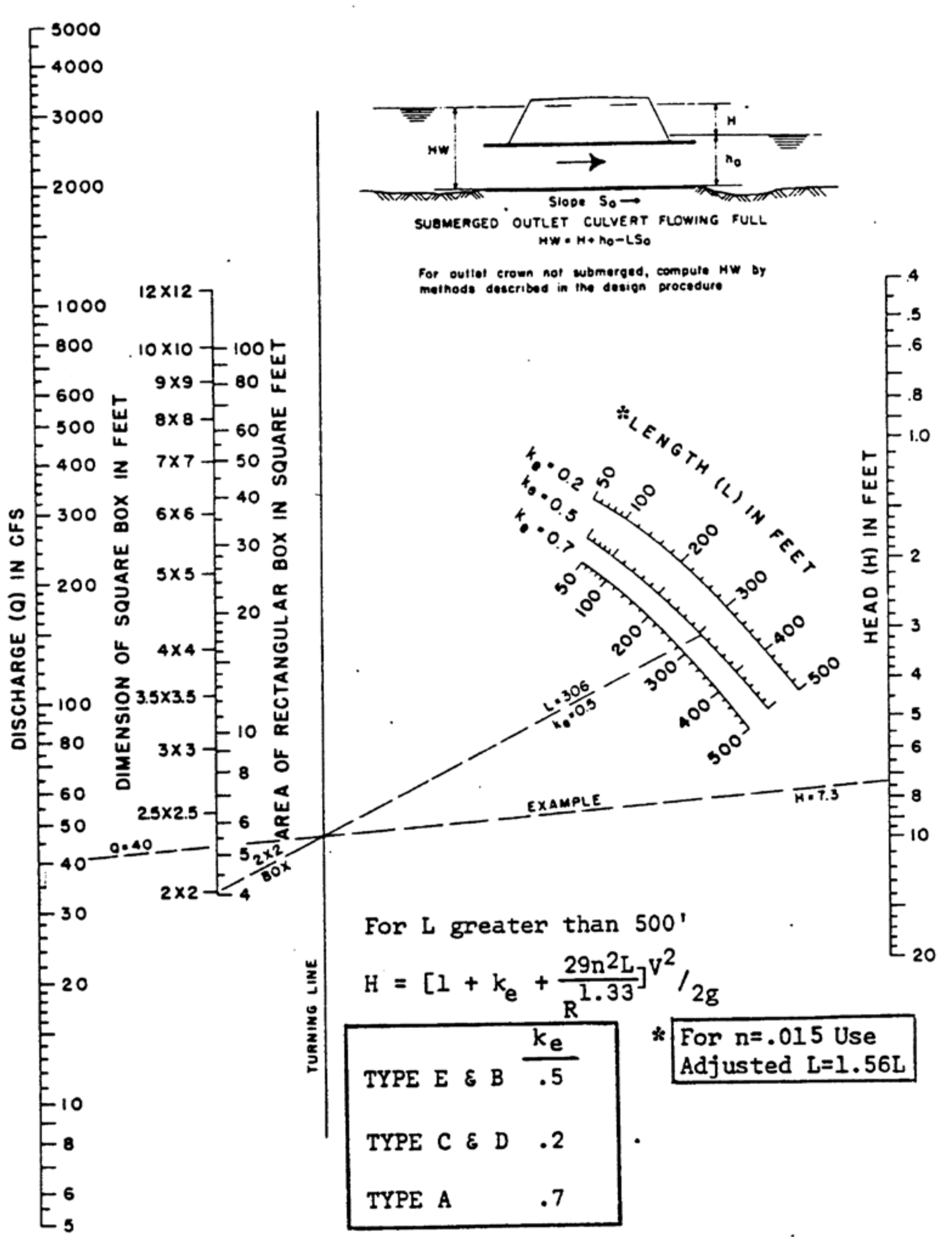
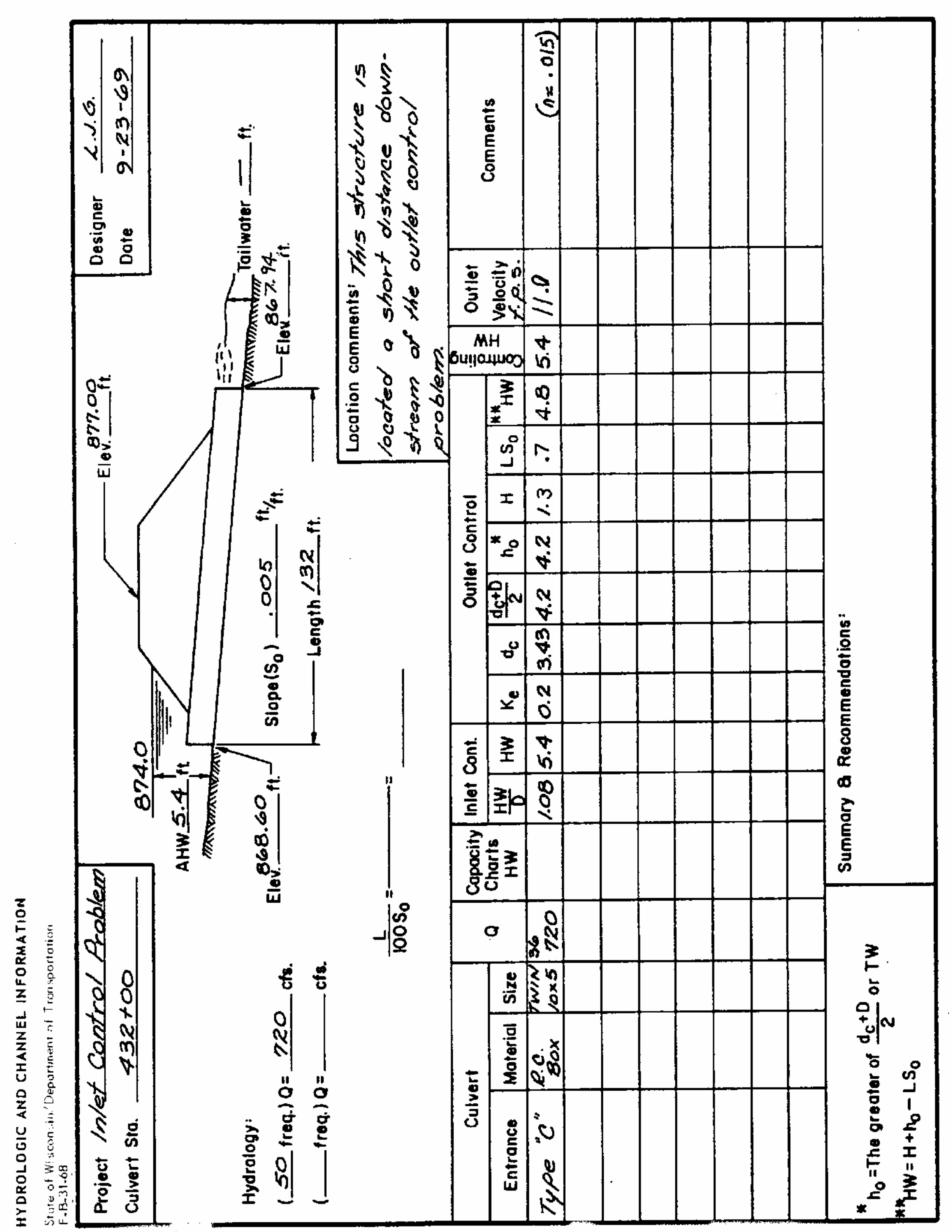


Figure 8.4-4

Head for Concrete Box Culverts Flowing Full, n = 0.012



HYDROLOGIC AND CHANNEL INFORMATION

State of Wisconsin/Department of Transportation
F-B-31-68

Project *Inlet Control Problem*
Culvert Sta. *432+00*

Designer *L.J.G.*
Date *9-23-69*

Elev. *877.00* ft.
874.0
AHW *5.4* ft
Tailwater ___ ft.
Elev. *868.60* ft.
Slope (S_o) *.005* ft/ft.
Elev. *867.94* ft.
Length *132* ft.

Hydrology:
(*50* freq.) Q = *720* cfs.
(___ freq.) Q = ___ cfs.

$\frac{L}{100 S_o}$ = ______ = ______

Location comments: *This structure is located a short distance down-stream of the outlet control problem.*

Culvert			Q	Capacity Charts HW	Inlet Cont.		Outlet Control							Controlling HW	Outlet Velocity f.p.s.	Comments
Entrance	Material	Size			HW/D	HW	K_e	d_c	$\frac{d_c+D}{2}$	h_o*	H	LS_o	**HW			
Type "C"	*R.C. Box*	*TWIN 10x5*	*36 720*		*1.08*	*5.4*	*0.2*	*3.43*	*4.2*	*4.2*	*1.3*	*.7*	*4.8*	*5.4*	*11.0*	*(n = .015)*

* h_o = The greater of $\frac{d_c+D}{2}$ or TW
** $HW = H + h_o - LS_o$

Summary & Recommendations:

Figure 8.4-5
Culvert Computation Form

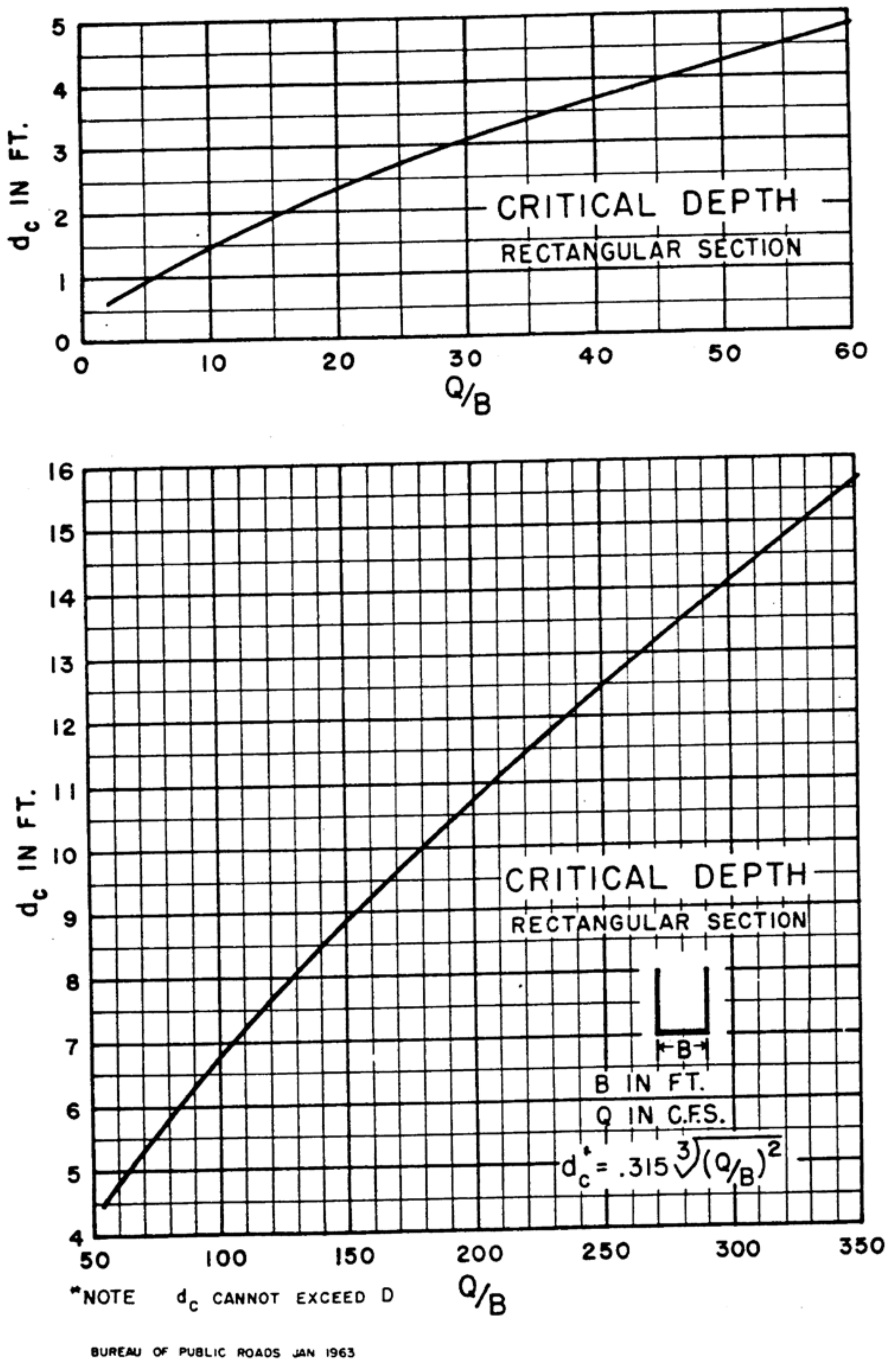


Figure 8.4-6
Critical Depth – Rectangular Section

8.4.2.6 Roadway Overflow

See 8.3.2.6.

8.4.2.7 Outlet Scour and Energy Dissipators

Energy dissipating devices are used where it is desirable to reduce the discharge velocity by inducing high energy losses at the inlet or discharge ends of the structure. They are generally warranted when discharge velocities exceed 14 feet per second.

Energy losses may be induced at the culvert entrance with a drop inlet, or at the outlet using energy dissipating devices and stilling basins to form a hydraulic jump.

Drop inlets are used where headroom is limited, and energy dissipating devices and stilling basins at the discharge are used where headroom is not critical.

The use of drop inlets should generally be reserved for areas where channel slopes are steep. Under these conditions drop inlets enable the reduction of culvert grades and in turn lower discharge velocities. When evaluating a site, a drop inlet may also be applicable on drainage ditches, in addition to channels that are normally dry or do not support fish or other aquatic organism habitat of pronounced significance. The use of a drop inlet requires approval from the Bureau of Structures, as well as coordination with the Department of Natural Resources early in project development.

For outlet devices utilizing the hydraulic jump, two conditions must be present for the formation of a hydraulic jump; the approach depth must be less than critical depth (supercritical flow); and the tailwater depth must be deeper than critical depth (subcritical flow) and of sufficient depth to control the location of the hydraulic jump. Where the tailwater depth is too low to cause a hydraulic jump at the desired location, the required depth can be provided by either depressing the discharge apron or utilizing a broad-crested weir at the end of the apron to provide a pool of sufficient depth. The depressed apron method is preferred since there is less scouring action at the end of the apron. The amount of depression is determined as the difference between the natural tailwater depth and the depth required to form a jump.

There are numerous design concepts of energy dissipating devices and stilling basins that may be adapted for energy dissipation to reduce the velocity and avoid scour at the culvert outlet. The more common type of designs are drop inlets, drop outlets, hydraulic jump stilling basins and riprap stilling basins.

More discussion on energy dissipators for culverts is available in 8.5 references (19), (20), (21), and (22). The designer is strongly advised to closely examine and study reference (20). More detailed discussions about the various types of energy dissipators and their designs are presented in that reference.

8.4.2.7.1 Drop Inlet.

In drop inlet design, flow is controlled at the inlet crest by the weir effect of the drop opening. Drop inlet culverts operate most satisfactorily when the height of drop is sufficient to permit considerable submergence of the culvert entrance without submerging the weir or exceeding limiting headwater depths.

Referring to Figure 8.4-7, the general formula for flow into the horizontal drop opening is:

$$Q = C_1 (2g)^{1/2} L H^{3/2}$$

Where Q is the discharge in c.f.s., L is the crest length 2B+W, H is the depth of flow plus velocity head, and C_1 is a dimensionless discharge coefficient taken as 0.4275. The formula is expressed in english units as:

$$Q = 3.43 LH^{3/2}$$

and

$$L = Q/(3.43H^{3/2})$$

There are four corrections which have to be multiplied times the discharge coefficient C_1, or times the factor 3.43:

1. Correction for head H/W (Table 8.4-1)

2. Correction for box-inlet shape B/W. (Table 8.4-2)

3. Correction for approach channel width W_c/L (Table 8.4-3).

   Where: W_c = approach channel width = Area/Depth

4. Correction for dike effect X/W (Table 8.4-4)

The size of the culvert should be determined by using the discharge (Q) and not allowing the height of water (HW) to exceed the inlet drop plus the critical depth of the weir which is given as:

$$d_c = [(Q/L)^2/g]^{1/3}$$

When using the hydraulic charts of 8.4.2.5, consider the culvert to have a wingwall flare of 0 degrees (extension of sides).

Sample computations are shown in 8.4.2.7.1.1.

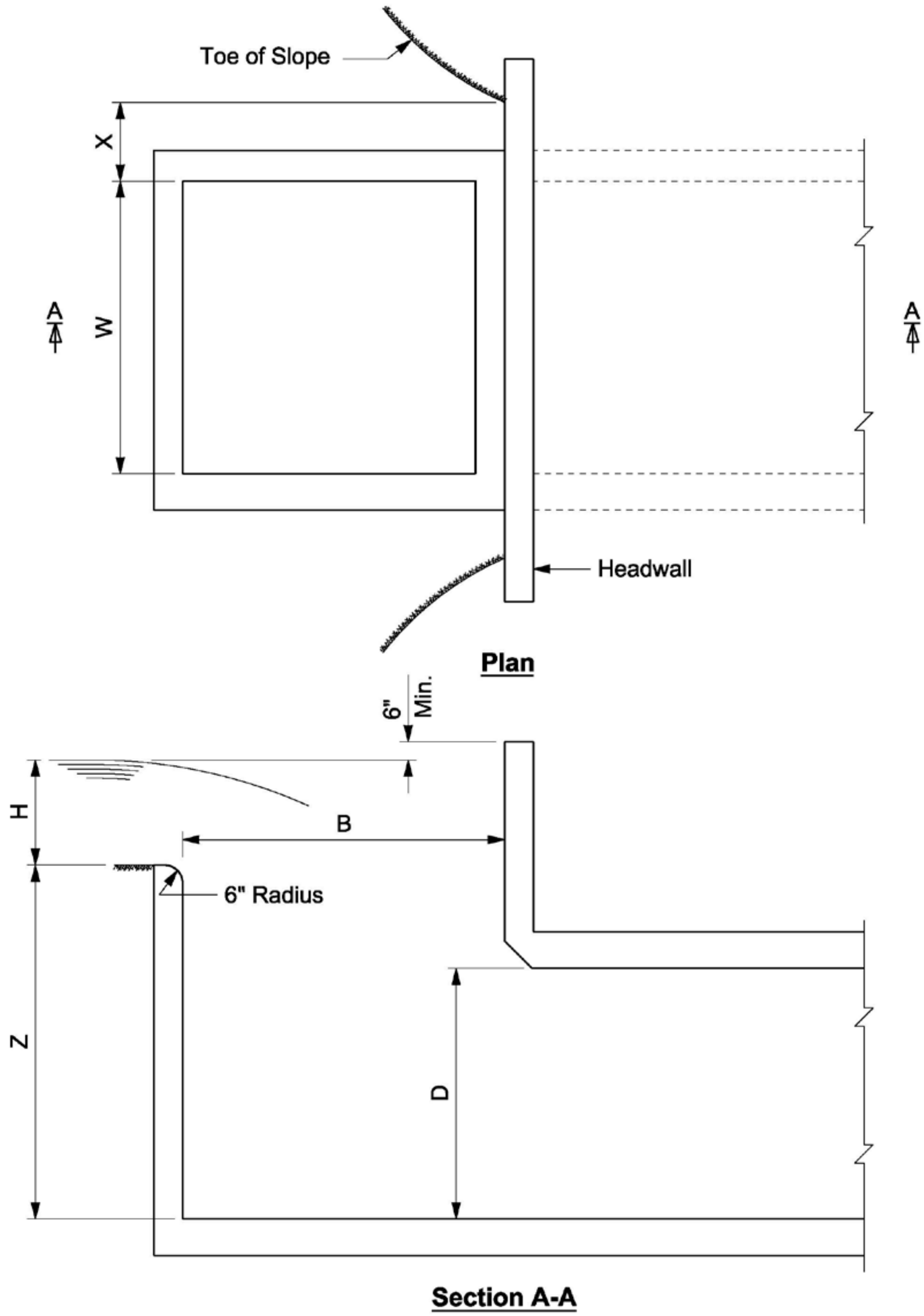


Figure 8.4-7
Box Drop Inlet

H/W	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	--	--	--	--	--	0.76	0.8	0.82	0.84	0.86
0.1	0.8	0.88	0.89	0.9	0.91	0.91	0.92	0.92	0.93	0.93
0.2	0.93	0.94	0.94	0.95	0.95	0.95	0.95	0.96	0.96	0.96
0.3	0.97	0.97	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.98
0.4	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	1
0.5	1	1	1	1	1	1	1	1	1	1
0.6	1	--	--	--	--	--	--	--	--	--
Correction is 1.00 when H/W exceeds 0.6										

Table 8.4-1
Correction for Head
(Control at Box-Inlet Crest)

B/W	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.98	1.01	1.03	1.03	1.04	1.04	1.03	1.02	1.01	1.01
1	1	0.99	0.99	0.98	0.98	0.98	0.97	0.97	0.96	0.96
2	0.96	0.96	0.95	0.95	0.95	0.95	0.95	0.95	0.94	0.94
3	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.93	0.93
4	0.93	--	--	--	--	--	--	--	--	--

Table 8.4-2
Correction for Box-Inlet Shape
(Control at Box-Inlet Crest)

Wc/L	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0.09	0.18	0.27	0.35	0.44	0.53	0.62	0.71	0.8
1	0.84	0.87	0.9	0.92	0.93	0.94	0.95	0.96	0.97	0.97
2	0.98	0.98	0.99	0.99	0.99	0.99	1	1	1	1
3	1	--	--	--	--	--	--	--	--	--
Correction is 1.00 when Wc/L exceeds 3.0										

Table 8.4-3
Correction for Approach-Channel Width
(Control at Box-Inlet Crest)

B/W	X/W						
	0	0.1	0.2	0.3	0.4	0.5	0.6
0.5	0.9	0.96	1	1.02	1.04	1.05	1.05
1	0.8	0.88	0.93	0.96	0.98	1	1.01
1.5	0.76	0.83	0.88	0.92	0.94	0.96	0.97
2	0.76	0.83	0.88	0.92	0.94	0.96	0.97

Table 8.4-4
Correction for Dike Effect
(Control at Box-Inlet Crest)

8.4.2.7.1.1 Drop Inlet Example Calculations

Given:

Q = 420 cfs through single 9'x6' box

H = 4.4' in a 27 ft. wide channel

Drop = 5 ft

Assume:

B = $\frac{W}{2} = 4.5$

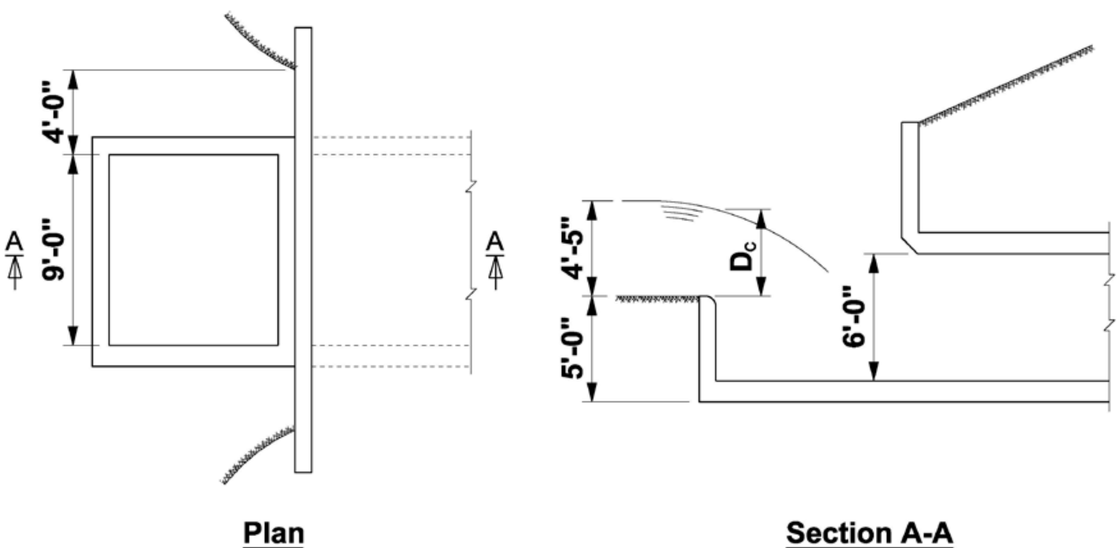


Figure 8.4-8
Drop Inlet Example

Control at inlet crest: $L = \dfrac{Q}{3.43 \cdot H^{3/2}}$

Corrections:

1. $\dfrac{H}{W} = \dfrac{4.4}{9} = 0.49 \Rightarrow 1.00$

2. $\dfrac{B}{W} = \dfrac{4.5}{9} = 0.5 \Rightarrow 1.04$

3. $\dfrac{W_c}{L} = \dfrac{27}{9 + 2(4.5)} = \dfrac{27}{18} = 1.50 \Rightarrow 0.94$

4. $\dfrac{X}{W} = \dfrac{4.0}{9.0} = 0.44 \Rightarrow 1.04$

Total Correction = 1.00 x 1.04 x 0.94 x 1.04 = 1.02

$$L = \frac{420}{1.02 \cdot 3.43 \cdot 4.4^{3/2}} = \frac{420}{1.02 \cdot 3.43 \cdot 9.23} = 13.01 < (2B + W) = 18 \Rightarrow OK$$

$$d_c = \sqrt[3]{\frac{Q^2}{L^2 g}} = \left(\frac{17.64x10^4}{3.24 \cdot 3.22x10^3}\right)^{1/3} = 16.85^{1/3} = 2.56$$

HW must be less than $Z+d_c$ to prevent submerged weir. With inlet control, from Figure 8.4-3:

$$\frac{HW}{D} = 1.19$$

HW = 1.19x6 = 7.14

7.14 < (5+2.56) = 7.56, therefore weir controls

8.4.2.7.2 Drop Outlets

This generalized design is applicable to relative heights of fall ranging from 1.0 y/d_c to 15 y/d_c and to crest lengths greater than 1.5 d_c. Here y is the vertical distance between the crest and the stilling basin floor and d_c is the critical depth of flow.

$d_c = 0.315[(Q/B)^2]^{1/3}$

Referring to Figure 8.4-10 and Figure 8.4-9, this design uses the following formulas:

1. The minimum length L_b of the stilling basin is:

$X_a + X_b + X_c = X_a + 2.55\ d_c$

a. The distance X_a from the headwall to the point where the surface of the upper nappe strikes the stilling basin floor is solved graphically in Figure 8.4-9.

b. The distance X_b from the point at which the surface of the upper nappe strikes the stilling basin floor to the upstream face of the floor blocks is:

$X_b = 0.8\ d_c$

c. The distance X_c, between the upstream face of the floor blocks and the end of the stilling basin is:

$X_c \geq 1.75\ d_c$

2. The floor blocks are proportioned as follows:

a. The height of the floor blocks is:

$0.8\ d_c$

b. The width and spacing of the floor blocks are approximately:

$0.4\ d_c$

A variation of $\pm$ 0.15 d_c from this limit is permissible.

c. The floor blocks are square in plan.

d. The floor blocks occupy between 50 and 60 percent of the stilling basin width.

3. The height of the end sill is:

$0.4\ d_c$

4. The sidewall height above the tailwater level is:

$0.85\ d_c$

5. The minimum height d_2, of the tailwater surface above the floor of the stilling basin is:

$d_2 = 2.15\ d_c$

In cases where the approach velocity head is greater than 1/3 of the specific head (velocity head + elevation head), X_a is checked by the formula below and the greater X_a value is used.

$$X_a^2 = \left(\frac{2 \cdot V^2}{g}\right) \cdot y_1$$

Where:

y_1 = top of water at crest

V = velocity of approach

Sometimes high values of d_c become unworkable, resulting in a need for large drops, high end sills and floor blocks. To prevent this d_c may be reduced by flaring the end of the barrel. The flare angle is approximately 150/V where V is the velocity at the beginning of the taper.

Sample computations are shown in 8.4.2.7.2.1.

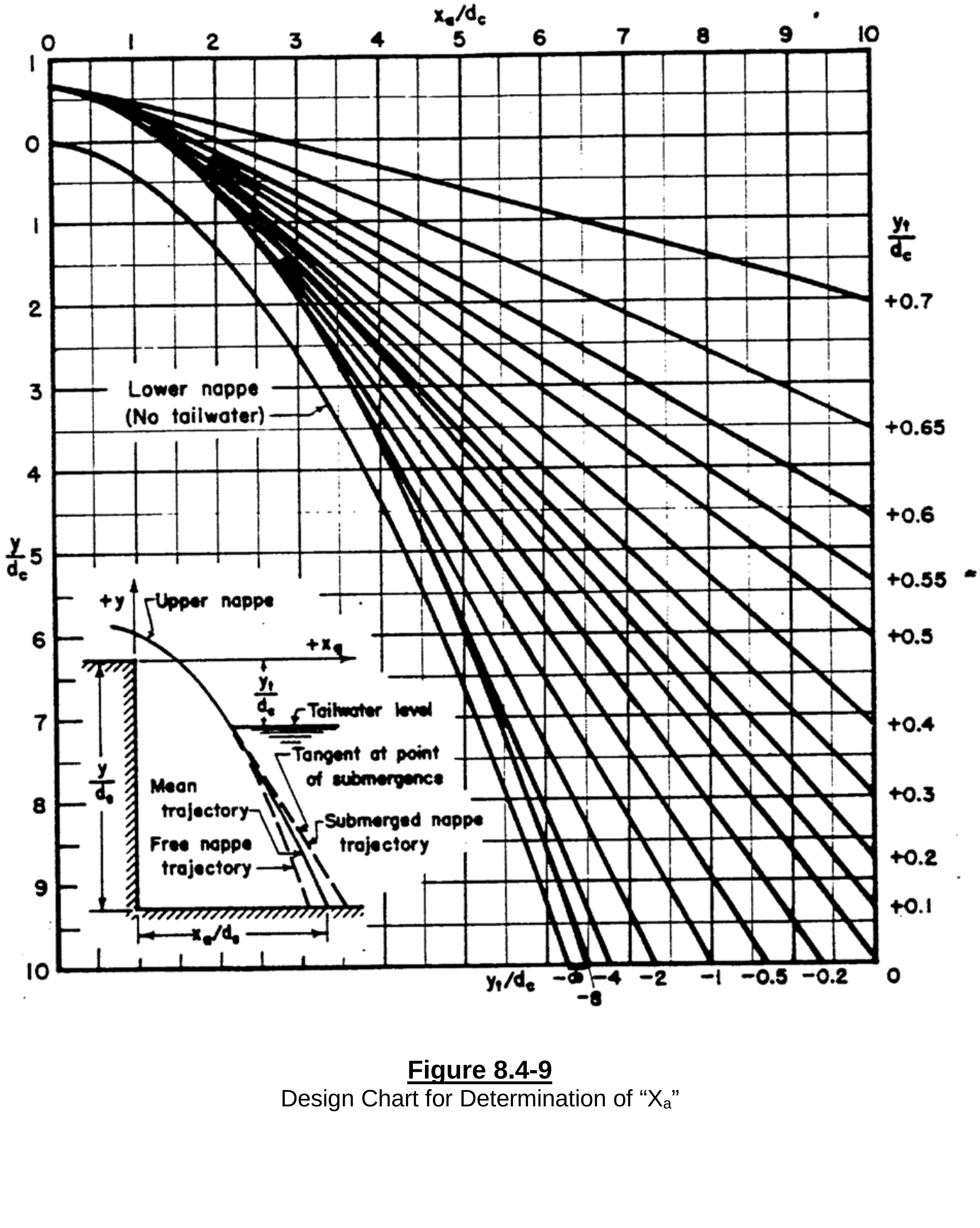


Figure 8.4-9
Design Chart for Determination of "X_a"

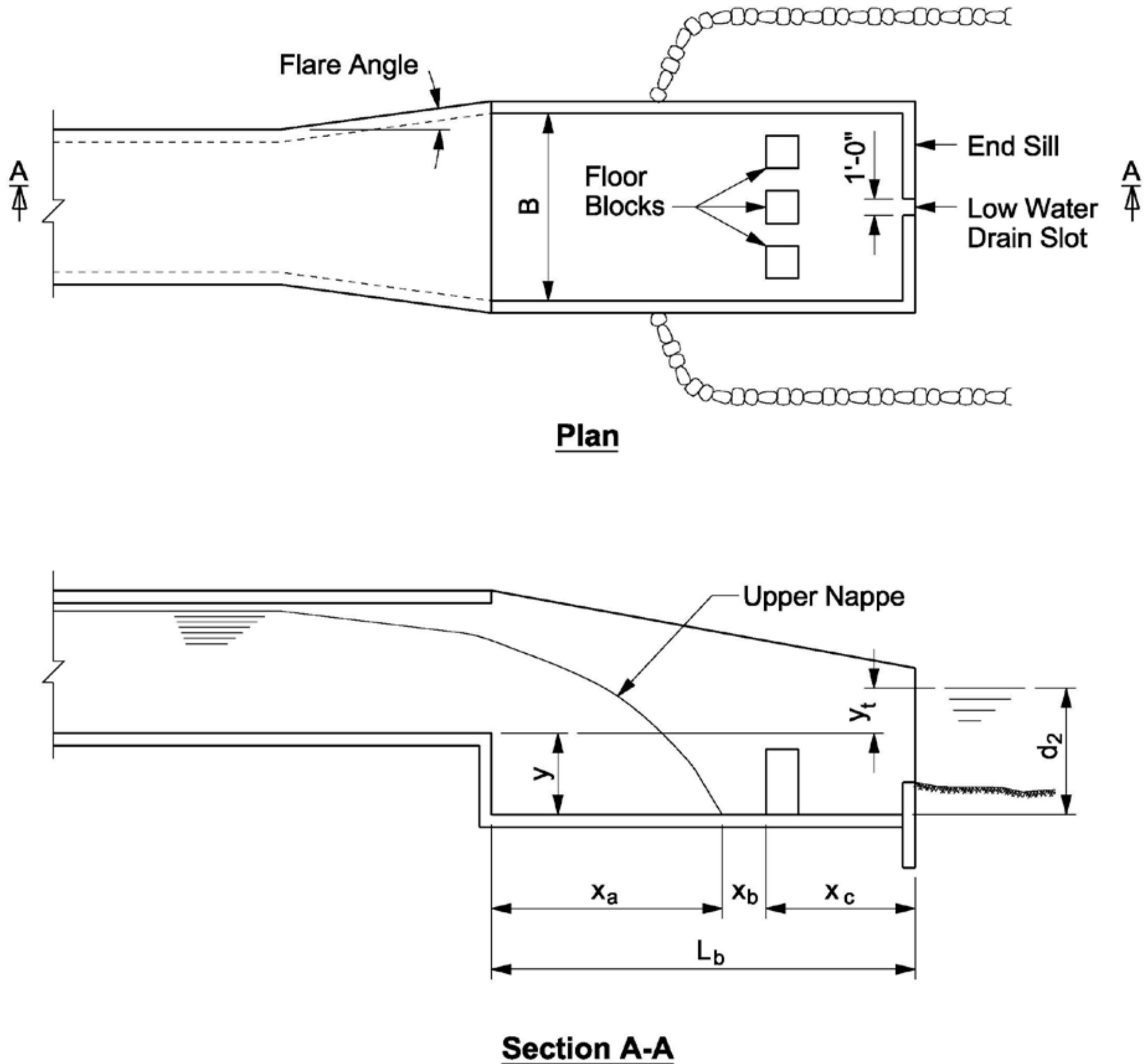


Figure 8.4-10
Straight Drop Outlet Stilling Basin

8.4.2.7.2.1 Drop Outlet Example Calculations

Given:

Q	=	800 cfs through single 8'x8' box
V	=	13.5 fps in the box
Drop	=	5 ft

Depth = 7.5 ft

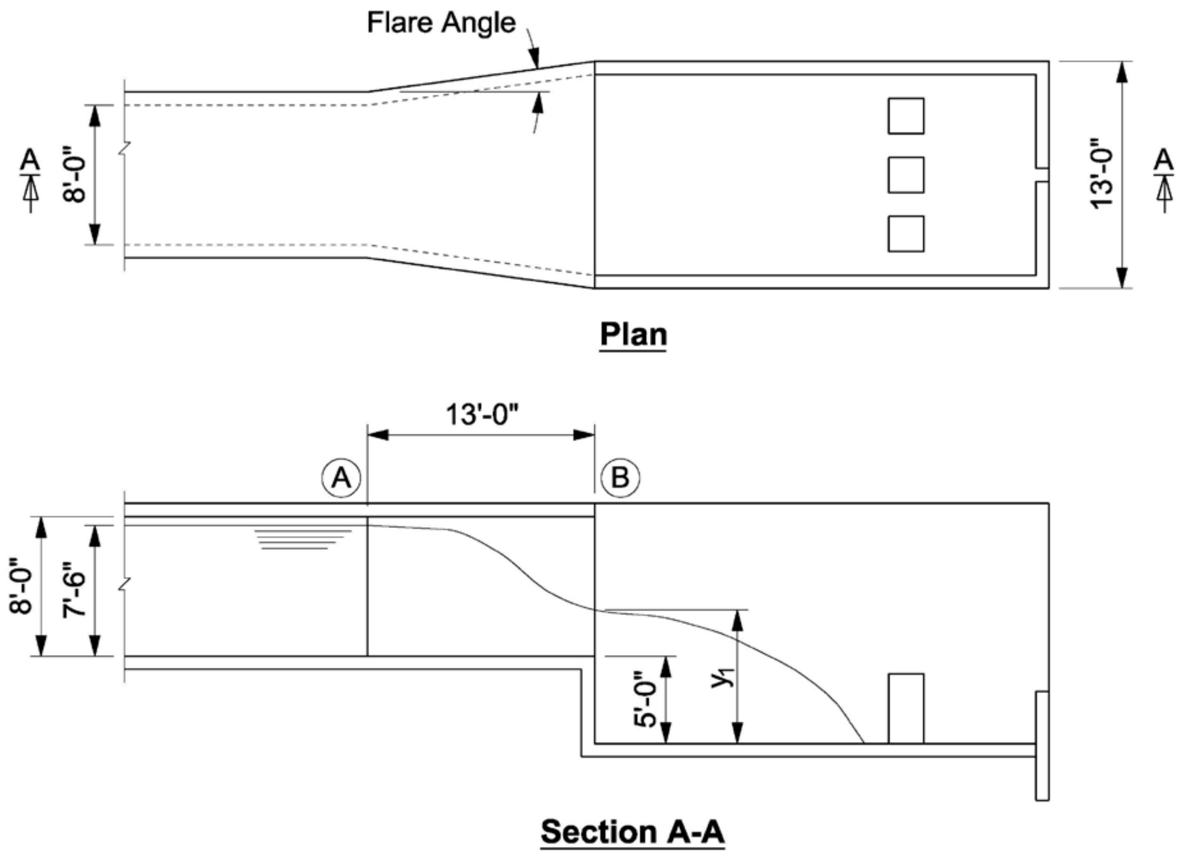


Figure 8.4-11
Drop Outlet Example

Assumptions:

- That the specific head of "A" is approximately equal to the specific head at "B". Therefore, the elevation head + velocity head at "A" = elevation head + velocity head at "B".
- The end sill height should be less than or equal to 2'-0".

If the drop were placed at "A":

$$d_c = 0.315 \cdot \sqrt[3]{\left(\frac{Q}{B}\right)^2} = 0.315 \cdot (100)^{2/3} = 6.78$$

And end sill = 0.4dc = 2'-9" which exceeds 2'-0, therefore flare outlet.

To obtain a 2'-0" sill, set d_c = 2'-0"/0.4 = 5 ft

$$B=\left(\frac{0.315\cdot Q^{2/3}}{d_c}\right)^{3/2}=\left(\frac{0.315\cdot 800^{2/3}}{5}\right)^{3/2}=13'$$

Flare from B = 9 ft to B = 13 ft at an angle of 150/13.5 = 11^0

$$\text{Length}=\frac{\left(\frac{13-8}{2}\right)}{\tan 11^0}=13'$$

$$\text{Specific Head, } H_A=7.5+\frac{V_A{}^2}{2g}=\frac{13.5^2}{2\cdot 32.2}=10.33'$$

By trial and error; assume $\frac{V_B{}^2}{2g}=7.5'$

$$V_B=(2\cdot 32.2\cdot 7.5)^{1/2}=22\text{fps}$$

Elevation head (depth) = 10.33-7.2 = 2.83’

Check trial; Q = AV = (13x2.83)x22 = 809 cfs, Q_{actual} = 800 cfs, OK

$$d_c=0.315\cdot\sqrt[3]{\left(\frac{Q}{B}\right)^2}=0.315\cdot\left(\frac{800}{13}\right)^{2/3}=0.315\cdot 15.6=4.91'$$

$$\frac{h_v}{H}=\frac{\left(\frac{V_B{}^2}{2g}\right)}{10.33}=\frac{7.5}{10.33}=0.725>\frac{1}{3} \qquad \therefore X_a{}^2=\frac{2V^2}{g}y_1$$

$$X_a=\left[\frac{2\cdot 22^2\cdot(5+2.83)}{32.2}\right]^{1/2}=15.35' \qquad \text{Use } X_a = 15'\text{-}6''$$

Dimensions:

Height of floor blocks	=	0.8 x 4.91 = 4’-0”
Height of end sill	=	0.4 x 4.91 = 2’-0”
Length of Basin	=	15.5+2.55 d_c = 28’
Floor Blocks	=	2’-0” square

Height of Sidewalls = $(2.15 + 0.85)d_c$ = 14.48' above basin floor. Use 13'-0"

8.4.2.7.3 Hydraulic Jump Stilling Basins

The simplest form of a hydraulic jump stilling basin has a straight centerline and is of uniform width. A sloping apron or a chute spillway is typically used to increase the Froude number as the water flows from the culvert to the stilling basin. The outlet barrel of the culvert is also sometimes flared to decrease y_1 so that the tailwater elevation necessary to cause a hydraulic jump need not be so high. This is done using the 150/V relationship as in the drop outlet sample problem. y_1 is usually kept in the 2-3 foot range.

Referring to Figure 8.4-12, the required tailwater is computed by the formula:

$$y_2/y_1 = \tfrac{1}{2}\ [(1+8F_1^2)^{1/2} - 1]$$

Where:

y_2 = tailwater height required to cause the hydraulic jump,

F_1 = Froude number = $v_1/(gy_1)^{1/2}$

g = acceleration of gravity,

y_1 = velocity at beginning of jump.

End sill height (ΔZ_0) is determined graphically from Figure 8.4-13

Length of jump is assumed to be 6 times the depth change (y_2-y_1).

In many cases the tailwater height isn't deep enough to cause the hydraulic jump. To remedy this, the slope of the culvert may be increased to greater than the slope of the streambed. This will result in an apron depressed such that normal tailwater is of sufficient depth.

The problem of scour on the downstream side of the end sill can be overcome by providing riprap in the stream bottom. If riprap is used, it starts from the top of the sill at a maximum slope of 6:1 up from end sill to original streambed. If no riprap is used, the streambed begins at the top of the end sill.

More detailed discussion about the various types of hydraulic jump stilling basins and their design can be found in 8.5 reference (20).

Sample computations are shown in 8.4.2.7.3.1.

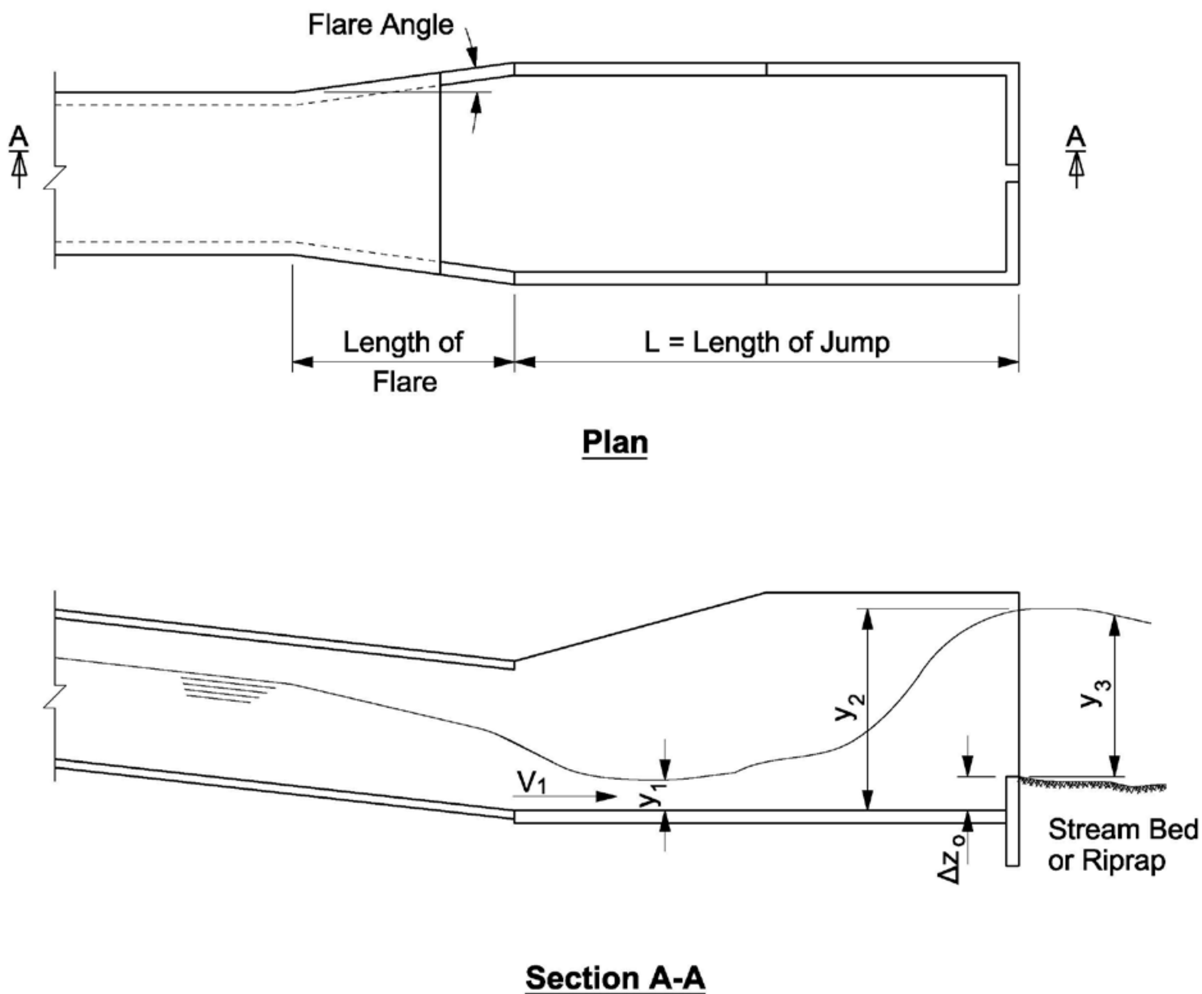


Figure 8.4-12
Hydraulic Jump Stilling Basin

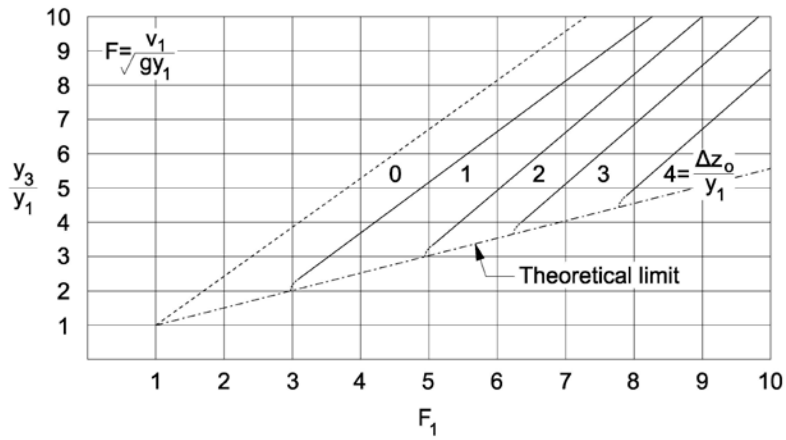


Figure 8.4-13
Characteristics of a Hydraulic Jump at an Abrupt Rise

8.4.2.7.3.1 Hydraulic Jump Stilling Basin Example Calculations

Given:

A discharge of 600 cfs flows through a 7'x6' box culvert at 16 fps and a depth of 5.8'. Normal tailwater depth in the outlet channel is 5.0 feet.

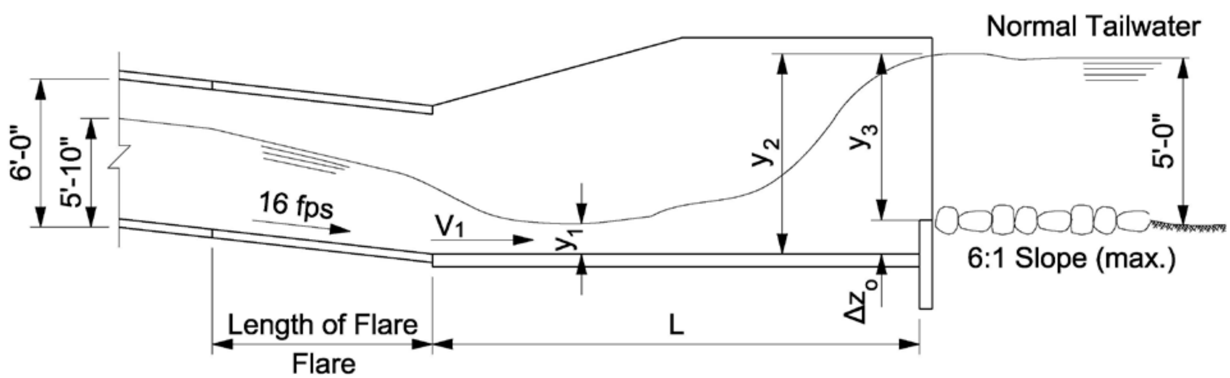


Figure 8.4-14
Hydraulic Jump Stilling Basin Example

Flare of wings $= \dfrac{150}{16} \approx 9^0$

$$H = 5.8 + \frac{16^2}{2 \times 32.2} = 5.8 + 3.975 = 9.775$$

Assume:

$y_1 = 2.2$ and $\frac{V_1^2}{2 \cdot g} = 9.775 - 2.2 = 7.575'$

$V_1 = (2 \times 32.2 \times 7.575)^{1/2} = 22.1$ fps

$Q = 600 = AV = 2.2 \times \text{width} \times 22.1$, width = 12.36

Length of flare = $\frac{\frac{(12.36 - 7)}{2}}{\tan 9^0} = 17'$

$Y_1 = 2.20$

$V_1 = 22.1$

$$F_1 = \frac{V_1}{\sqrt{g \cdot y_1}} = \frac{22.1}{\sqrt{32.2 \times 2.2}} = 2.63$$

$$y_2 = y_1 \cdot \frac{1}{2} \cdot \left(\sqrt{1 + 8 \times 2.63^2} - 1\right) = 7.15$$

$L = 6(y_2 – y_1) = 6 (7.15 – 2.20) = 29.7'$ use L = 30 ft.

Assume $y_3 = 5'$

$y_3/y_1 = 5/2.2 = 2.27$

From Figure 8.4-13, $\Delta Z_o/y_1 = 0.5$

$\Delta Z_o = 1.1$, use 1'-6"

8.4.2.7.4 Riprap Stilling Basins

The riprap stilling basins, in many cases, is a very economical approach to dissipate energy at culvert outlets and avoid damaging scour. A good treatise on riprap stilling basin is given in the FHWA Hydraulic Design of Energy Dissipators for Culverts and Channels, see 8.5 reference (20).

8.4.2.8 Select Culvert Design Alternatives

The "proposed culvert" design shall be based on several design factors. In most design situations, the pertinent hydraulic factors discussed in 8.4.1 will dictate the final selection of culvert size, length, scour protection, as well as the approach roadway design.

8.5 References

1. Wisconsin Department of Natural Resources, *Wisconsin's Floodplain Management Program, Chapter NR116*, Register, August 2004, No. 584.

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   http://wi.water.usgs.gov/publications/flood/currentreport.html

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9. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Applications Guide* (CPD-70), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

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11. L.A. Arneson and J.O. Shearman, *User's Manual for WSPRO-A computer Model for Water Surface Profile Computations,* FHWA Report No. FHWA-SA-98-080, June 1998.

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16. U.S. Department of Transportation, Federal Highway Administration, *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, Office of Engineering, Bridge Division, Report No. FHWA-PD-96-001, December 1995.

17. U.S. Department of Transportation, Federal Highway Administration, *Highways in the River Environment,* Report No. FHWA-HI-90-016, February 1990.

18. U.S. Department of Transportation, FHWA, *Debris-Control Structures, Evaluation and Countermeasures, Third Edition,* Hydraulic Engineering Circular (HEC) No.9, Publication No. FHWA-IF-014-016, October 2005.

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20. U.S. Department of Transportation, Federal Highway Administration, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular (HEC) No. 14, Third Edition, Publication No. FHWA-NHI-06-086, July 2006.

21. Blaisdell, Fred W. and Donnelly, Charles A., *Hydraulic Design of the Box Inlet Drop Spillway*, U.S. Department of Agriculture, Soil Conservation Service, SCS-TP-106, July, 1951.

22. Blaisdell, Fred W. and Donnelly, Charles A., *Straight Drop Spillway Stilling Basin*, University of Minnesota, St. Anthony Falls Hydraulic Laboratory, November, 1954.

8.6 Appendix 8-A, Check List for Hydraulic/Site Report

A hydraulic and site report shall be prepared for all stream crossing bridge and culvert projects that are completed by consultants. The report shall be submitted to the Bureau of Structures for review along with the "Stream Crossing Structure Survey Report" and preliminary structure plans (see WisDOT Bridge Manual, 6.2.1). The hydraulic and site report needs to include information necessary for the review of the hydraulic analysis and the type, size and location of proposed structure. The following is a list of the items that need to be included in the hydraulic site report:

- Document the location of the stream crossing or project site. Indicate county, municipality, Section, Town, and Range.

- List available information and references for methodologies used in the report. Indicate when survey information was collected and what vertical datum was used as reference for elevations used in hydraulic models and shown on structure plans. Indicate whether the site is in a mapped flood hazard area and type of that mapping, if any.

- Provide complete description of the site, including description of the drainage basin, river reach upstream and downstream of the site, channel at site, surrounding bank and over bank areas, and gradient or slope of the river. Also, provide complete description of upstream and downstream structures.

- Provide a summary discussion of the magnitude and frequency of floods to be used for design. Hydrologic calculations shall be provided to the Bureau of Structures beforehand for their review and concurrence. Indicate in the hydraulic site report when calculations were submitted and whether approval was obtained.

- Provide a description of the hydraulic analyses performed for the project. Indicate what models were used and the basis for and assumptions used in the selection of various modeling parameters. Specifically, discuss the assumptions used for defining the modeling reach boundary conditions, roughness coefficients, location and source of hydraulic cross sections, and any assumptions made in selecting the bridge modeling methodology. (Hydraulic calculations shall be submitted with the hydraulic site report).

- Provide a complete description of the existing structure, including a description of the geometry, type, size and material. Indicate the sufficiency rating of the structure. Provide information about observed scour, flooding, roadway overtopping, ice or debris, navigation clearance and any other structurally or hydraulically pertinent information. Provide a discussion of calculated hydraulic characteristics at the site.

- Provide a description of the various sizing constraints considered at the site, including but not limited to regulatory requirements, hydraulic and roadway geometric conditions, environmental and constructability considerations, etc.

- Provide a discussion of the alternatives considered for this project including explanations of how certain alternatives are removed from consideration and how the recommended alternative is selected. Include a cost comparison.

- Provide complete description of proposed structure including calculated hydraulic characteristics.

- Provide a discussion of calculated scour depths, recommended scour prevention measures and assigned scour code. (Scour calculations shall be submitted with the hydraulic site report).

- Provide a summary table comparing calculated hydraulic characteristics for existing and proposed conditions.

8.7 Appendix 8-B, FHWA Hydraulic Engineering Publications

Note: Some links may be obsolete, but will be updated in the future.

Code	Title	Year	Publication #	NTIS #
HDS 01	Hydraulics of Bridge Waterways	1978	FHWA-EPD-86-101	PB86-181708
HDS 02	Highway Hydrology Second Edition	2002	FHWA-NHI-02-001	
HDS 03	Design Charts for Open-Channel Flow	1961	FHWA-EPD-86-102	PB86-179249
HDS 04	Introduction to Highway Hydraulics	2001	FHWA-NHI-01-019	
HDS 05	Hydraulic Design of Highway Culverts	2005	FHWA-NHI-01-020	
HDS 06	River Engineering for Highway Encroachments	2001	FHWA-NHI-01-004	
HEC 09	Debris Control Structures Evaluation and Countermeasures	2005	FHWA-IF-04-016	
HEC 11	Design of Riprap Revetment	1989	FHWA-IP-89-016	PB89-218424
HEC 14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2006	FHWA-NHI-06-086	
HEC 15	Design of Roadside Channels with Flexible Linings, Third Edition	2005	FHWA-IF-05-114	
HEC 17	The Design of Encroachments on Flood Plains Using Risk Analysis	1981	FHWA-EPD-86-112	PB86-182110
HEC 18	Evaluating Scour at Bridges, Fourth Edition	2001	FHWA-NHI-01-001	
HEC 18	Evaluating Scour at Bridges, Fourth Edition (Errata Sheet)	2001		
HEC 20	Stream Stability at Highway Structures Third Edition	2001	FHWA-NHI-01-002	
HEC 20	Stream Stability at Highway Structures Third Edition (Errata Sheet)	2001	FHWA-NHI-01-002	
HEC 21	Bridge Deck Drainage Systems	1993	FHWA-SA-92-010	PB94-109584
HEC 22	Urban Drainage Design Manual Second Edition	2001	FHWA-NHI-01-021	
HEC 23	Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Second Edition	2001	FHWA-NHI-01-003	
HEC 23	Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Second Edition (Errata Sheet)	2001		
HEC 24	Highway Stormwater Pump Station Design (cover)	2001	FHWA-NHI-01-007	
HEC 24	Highway Stormwater Pump Station Design	2001	FHWA-NHI-01-007	
HEC 25	Tidal Hydrology, Hydraulics, and Scour at Bridges	2004	FHWA-NHI-05-077	
HEC 25	Highways in the Coastal Environment - 2nd edition	2008	FHWA-NHI-07-096	
HRT	Assessing Stream Channel Stability at Bridges in Physiographic Regions	2006	FHWA-HRT-05-072	

Code	Title	Year	Publication #	NTIS #
HRT	Effects of Inlet Geometry on Hydraulic Performance of Box Culverts	2006	FHWA-HRT-06-138	
HRT	Junction Loss Experiments: Laboratory Report	2007	FHWA-HRT-07-036	
HRT	Hydraulics Laboratory Fact Sheet	2007	FHWA-HRT-07-054	
Other	Geosynthetic Design and Construction Guidelines	1995	FHWA-HI-95-038	PB95-270500
Other	Underwater Evaluation And Repair of Bridge Components	1998	FHWA-DP-98-1	
Other	Best Management Practices for Erosion and Sediment Control	1995	FHWA-FLP-94-005	
Other	Underwater Inspection of Bridges	1980	FHWA-DP-80-1	
Other	Culvert Management Systems User Manual	2001	FHWA-02-001	
Other	FHWA Hydraulics Library on CD-ROM FHWA Hydraulics Library on CD-ROM (Updated Browser)	2002		
Other	Hydraulic Performance of Curb and Gutter Inlets	1999	FHWA-KU-99-1	
Other	Culvert Management Systems Source Code	2001		
Other	NCHRP Report 25-25 (04) Environmental Stewardship Practices, Procedures, and Policies for Highway Construction and Maintenance	2004		
Other	New England Transportation Consortium: Performance Specs for Wood Waste Materials as an Erosion Control Mulch and as a Filter Berm	2001	FHWA-NETC 25	
Other	Bridge Scour Protection Systems Using Toskanes	1994	FHWA-PA-94-012	PB95-266318
Other	Structural Design Manual for Improved Inlets and Culverts	1983	FHWA-IP-83-6	PB84-153485
Other	Culvert Inspection Manual	1986	FHWA-IP-86-2	PB87-151809
RD	Bottomless Culvert Scour Study: Phase II Laboratory Report	2007	FHWA-HRT-07-026	
RD	Effects of Gradation and Cohesion on Scour, Volume 2, "Experimental Study of Sediment Gradation and Flow Hydrograph Effects on Clear Water Scour Around Circular Piers"	1999	FHWA-RD-99-184	PB2000-103271
RD	Effects of Gradation and Cohesion on Scour, Volume 1, "Effect of Sediment Gradation and Coarse Material Fraction on Clear Water Scour Around Bridge Piers"	1999	FHWA-RD-99-183	PB2000-103270
RD	Portable Instrumentation for Real Time Measurement of Scour At Bridges	1999	FHWA-RD-99-085	PB2000-102040
RD	Users Primer for BRI-STARS	1999	FHWA-RD-99-191	PB2000-107371

Code	Title	Year	Publication #	NTIS #
RD	Effects of Gradation and Cohesion on Scour, Volume 3, "Abutment Scour for Nonuniform Mixtures"	1999	FHWA-RD-99-185	PB2000-103272
RD	Remote Methods of Underwater Inspection of Bridge Structures	1999	FHWA-RD-99-100	PB9915-7968
RD	Hydraulics of Iowa DOT Slope-Tapered Pipe Culverts	2001	FHWA-RD-01-077	
RD	Users Manual for BRI-STARS	1999	FHWA-RD-99-190	PB2000-107372
RD	Effects of Gradation and Cohesion on Scour, Volume 4, "Experimental Study of Scour Around Circular Piers in Cohesive Soils"	1999	FHWA-RD-99-186	PB2000-103273
RD	Effects of Gradation and Cohesion on Scour, Volume 5, "Effect of Cohesion on Bridge Abutment Scour"	1999	FHWA-RD-99-187	PB2000-103274
RD	Effects of Gradation and Cohesion on Scour, Volume 6, "Abutment Scour in Uniform and Stratified Sand Mixtures"	1999	FHWA-RD-99-188	PB2000-103275
RD	Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe	2000	FHWA-RD-97-140	
RD	Performance Curve for a Prototype of Two Large Culverts in Series Dale Boulevard, Dale City, Virginia	2001	FHWA-RD-01-095	
RD	Bottomless Culvert Scour Study: Phase I Laboratory Report	2002	FHWA-RD-02-078	
RD	Bridge Scour in Nonuniform Sediment Mixtures and in Cohesive Materials: Synthesis Report	2003	FHWA-RD-03-083	PB-2204-104690
RD	Enhanced Abutment Scour Studies For Compound Channels	2004	FHWA-RD-99-156	
RD	Field Observations and Evaluations of Streambed Scour at Bridges	2005	FHWA-RD-03-052	
RD	South Dakota Culvert Inlet Design Coefficients	1999	FHWA-RD-01-076	

Figure 8.7-1
FHWA Hydraulic Engineering Publications

FHWA Hydraulics Engineering Software		
Software	Title	Year
HY 7	Bridge Waterways Analysis Model (WSPRO)	2005
HY 7	WSPRO User's Manual (Version 061698) (pdf 2.1 MB)	1998
HY 8	Culvert Hydraulic Analysis Program, Version 7.0	2007
HDS 5	HDS 5 Hydraulic Design of Highway Culverts (pdf, 9.25 mb)	2001
HDS 5	HDS 5 Chart Calculator	2001
HY 11	Preliminary Analysis System for WSP	1989
HY 11	PAS USERS MANUAL (ISDDC)	1989
HY 11	Accuracy of Computed Water Surface Profiles (ISDDC)	1986
FESWMS	FESWMS (Version 3.1.5)	2003
FESWMS	FESWMS User's Manual	2003
HY 22	Visual Urban	2002
HY 22	HEC 22 - Urban Drainage Manual	2001
BRI-STARS	Bridge Stream Tube for Alluvial River Sim	2000
BRI-STARS	BRI-STARS Users Manual	2000
HYRISK	HYRISK Setup (zip, 13 mb)	2002
Hydraulics Software by Others		
Software	Title	Year
BCAP	Broken-back Culvert Analysis Program (Version 3.0)	2002
CAESAR	Cataloging And Expert evaluation of Scour risk And River stability at bridge sites	2001
CHL	Coastal & Hydraulics Laboratory USACE	
FishXing	Fish Passage through Culverts USFS	
HEC	Hydrologic Engineering Center USACE	
HyperCalc	HyperCalc Plus	2002
NSS	National Streamflow Statistics Program	
PEAKFQ	PEAKFQ	1995
SMS	Surface-Water Modeling System (SMS)	2001
StreamStats	StreamStats	
USGS	Water Resources Applications Software USGS	
WMS	Watershed Modeling System (WMS)	

Figure 8.7-2
FHWA Hydraulics Software List

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Table of Contents

9.1 General

The *Wisconsin Standard Specifications for Highway and Structure Construction* (hereafter referred to as *Standard Specifications*) contains references to *ASTM Specifications* or *AASHTO Material Specifications* which provide required properties and testing standards for materials used in highway structures. The service life of a structure is dependent upon the quality of the materials used in its construction as well as the method of construction. This chapter highlights applications of materials for highway structures and their properties.

In cases where proprietary products are experimentally specified, special provisions are written which provide material properties and installation procedures. Manufacturer's recommendations for materials, preparation and their assistance during installation may also be specified.

Materials that are proposed for incorporation into highway structure projects performed under the jurisdiction of the Wisconsin Department of Transportation (WisDOT) may be approved or accepted by a variety of procedures:

- Laboratory testing of materials that are submitted or samples randomly selected.
- Inspection and/or testing at the source of fabrication or manufacture.
- Inspection and/or testing in the field by WisDOT regional personnel.
- Manufacturer's certificate of compliance and/or manufacturer's certified report of test or analysis, either as sole documentation for acceptance or as supplemental documentation.
- Inspection, evaluation and testing in the normal course of project administration of material specifications.
- Some products are on approved lists or from fabricators, manufacturers, and certified sources approved by WisDOT. Lists of approved suppliers, products, and certified sources are located at:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

The *Wisconsin Construction and Materials Manual* (CMM) contains a description of procedures for material testing and acceptance requirements in Chapter 8, Section 45. Materials, unless otherwise permitted by the specifications, cannot be incorporated in the project until tested and approved by the proper authority.

9.2 Concrete

Concrete is used in many highway structures throughout Wisconsin. Some structure types are composed entirely of concrete, while others have concrete members. Different concrete compressive strengths (f'_c) are used in design and depend on the structure type or the location of the member. Compressive strengths are verified by cylinder tests done on concrete samples taken in the field. The *Standard Specifications* describe the requirements for concrete in Section 501.

Some of the concrete structure types/members and their design strengths for new projects are:

- Decks, Diaphragms, Overlays, Curbs, Parapets, Medians, Sidewalks and Concrete Slab Bridges (f'_c = 4 ksi)
- Other cast-in-place structures such as Culverts, Cantilever Retaining Walls and Substructure units (f'_c = 3.5 ksi)
- Other types of Retaining Walls (f'_c - values as specified in Chapter 14)
- Prestressed "I" Girders (f'_c = 6 to 8 ksi)
- Prestressed Box Girders (f'_c = 5 ksi)
- Prestressed Deck Panels (f'_c = 6 ksi)

Grade "E" concrete (Low Slump Concrete) is used in overlays for decks and slabs as stated in Section 509.2.

The modulus of elasticity of concrete, E_c, is a function of the unit weight of concrete and its compressive strength **LRFD [C5.4.2.4]**. For a unit weight of 0.150 kcf, the modulus of elasticity is:

f'_c = 3.5 ksi ; E_c = 3600 ksi

f'_c = 4 ksi ; E_c = 3800 ksi

For prestressed concrete members, the value for E_c is based on studies in the field and is calculated as shown in 19.3.3.8.

The modulus of rupture for concrete, f_r, is a function of concrete strength and concrete density, and is described in **LRFD [5.4.2.6]**. The coefficient of thermal expansion for normal weight concrete is 6×10^{-6} in/in/°F per **LRFD [5.4.2.2]**.

Air entraining admixture is added to concrete to provide durability for exposure to freeze and thaw conditions. Other concrete admixtures used are set retarding and water reducing admixtures. These are covered in Section 501 of the *Standard Specifications.*

9.3 Reinforcement Bars

Reinforced concrete structures and concrete members are designed using Grade 60 deformed bar steel with a minimum yield strength of 60 ksi. The modulus of elasticity, E_s , for steel reinforcing is 29,000 ksi. Reinforcement may be epoxy coated and this is determined by its location in the structure as described below. Adequate concrete cover and epoxy coating of reinforcement contribute to the durability of the reinforced concrete structure. The *Standard Specifications* describe the requirements for steel reinforcement and epoxy coating in Section 505.

Epoxy coated bars shall be used for both top and bottom reinforcement on all new decks, deck replacements, concrete slab superstructures, structural approach slabs and top slab of culverts (with no fill on top). They shall be used in other superstructure elements such as curbs, parapets, medians, sidewalks, diaphragms and pilasters. Some of the bars in prestressed girders are epoxy coated and are specified in the Chapter 19 - Standards. Also use coated bars for sign bridge footings.

Use epoxy coated bar steel on all piers detailed with expansion joints and on all piers at grade separations. Use epoxy coated bars down to the top of the footing elevation.

At all abutments, epoxy coated bars shall be used for parapets on wing walls. For A3/A4 abutments use epoxy coated bars for the paving block and the abutment backwall, and for A1(fixed) coat the dowel bars. For all abutments use epoxy coated bars in the wing walls.

Welding of bar steel is not permitted unless approved by the Bureau of Structures or used in an approved butt splice as stated in Section 505.3.3.3 of the *Standard Specifications*. Test results indicate that the fatigue life of steel reinforcement is reduced by welding to them. Supporting a deck joint by welding attachments to the bar steel is not permitted. The bar steel mat does not provide adequate stiffness to support deck joints or similar details during the deck pour and maintain the proper joint elevations.

The minimum and maximum spacing of reinforcement, and spacing between bar layers is provided in **LRFD [5.10.3.1, 5.10.3.2]**. Use minimum and maximum values shown on Standards where provided.

Bridge plans show the quantity of bar steel required for the structure. Details are not provided for bar chairs or other devices necessary to support the reinforcement during the placement of the concrete. This information is covered by the *Standard Specifications* in Section 505.3.4 and these devices are part of the bid quantity.

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete as stated in **LRFD [5.10.8]**.

When determining the anchorage requirements for bars, consider the bar size, the development length for straight bars and the development length for standard hooks. Note in Table 9.9-1 and Table 9.9-2 that smaller bars require considerably less development length than larger bars and the development length is also less if the bar spacing is 6 inches or more. By detailing smaller bars to get the required area and providing a spacing of 6 inches or more, less steel is used. Bar hooks can reduce the required bar development lengths, however the hooks may cost more to fabricate. In cases such as footings for columns or retaining walls, hooks may be the only practical solution because of the concrete depth available for developing the capacity of the bars.

Fabricators stock all bar sizes in 60 foot lengths. For ease of handling, the detailed length for #3 and #4 bars is limited to 45 feet. Longer bars may be used for these bar sizes at the discretion of the designer, when larger quantities are required for the structure. All other bar sizes are detailed to a length not to exceed 60 feet, except for vertical bars. Bars placed in a vertical position are detailed to match optional construction joint spacing plus lap. The location of optional horizontal construction joints in pier shafts or columns will generally determine the length of vertical bars in piers. All bars are detailed to the nearest inch.

The number of bars in a bundle shall not exceed four, except in flexural members the bars larger than #11 shall not exceed two in any one bundle. Individual bars in a bundle, cut off within the span of a member, shall be terminated at different points with at least a 40-bar diameter stagger. Where spacing limitations are based on bar size, bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area **LRFD [5.10.3.1.5]**.

Stainless steel deformed reinforcement meeting the requirements of ASTM A955 has been used on a limited basis with the approval of the Bureau of Structures. It has been used in bridge decks, parapets and in the structural approach slabs at the ends of the bridge. Fabricators typically stock #6 bars and smaller in 60 foot lengths and #7 bars and larger in 40 foot lengths. Follow the guidance above for selecting bar lengths based on ease of handling.

9.3.1 Development Length and Lap Splices for Deformed Bars

Table 9.9-1 and Table 9.9-2 provide the development length, ℓ_d , for straight bars and the required lap length of spliced tension bars according to **LRFD [5.11.2.1, 5.11.5.3]**. The basic development length, ℓ_{db} , is a function of bar area, A_b , bar diameter, d_b , concrete strength, f'_c and yield strength of reinforcement, f_y. The basic development length is multiplied by applicable modification factors to produce the required development length, ℓ_d. The lap lengths for spliced tension bars are equal to a factor multiplied times the development length, ℓ_d . The factor applied depends on the classification of the splice; Class A, B or C. The class selected is a function of the numbers of bars spliced at a given location and the ratio of the area of reinforcement provided to the area required. The values for development length (required embedment) are equal to Class "A" splice lengths shown in these tables. Table 9.9-1 gives the development lengths and required lap lengths for a concrete compressive strength of f'_c = 3.5 ksi and a reinforcement yield strength of f_y = 60 ksi. Table 9.9-2 gives these same lengths for a concrete compressive strength of f'_c = 4 ksi and a reinforcement yield strength of f_y = 60 ksi. In tensile stress zones the maximum allowable change in bar size at a lap is three bar sizes. The spacing of lap splice reinforcement is provided in **LRFD [5.10.3.1.4]**, but values on Standards should be used where provided.

The development length of individual bars within a bundle, shall be that for the individual bar, increased by 20% for a three-bar bundle and by 33% for a four-bar bundle **LRFD [5.11.2.3]**. For determining the modification factors specified in **LRFD [5.11.2.1.2, 5.11.2.1.3]**, a unit of bundled bars shall be treated as a single bar of a diameter determined from the equivalent total area.

Lap splices within bundles shall be as specified in **LRFD [5.11.2.3]**. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced **LRFD [5.11.5.2.1]**.

Hook and embedment requirements for transverse (shear) reinforcement are stated in **LRFD [5.11.2.6.2]**. The lap length for pairs of U-stirrups or ties that are placed to form a closed unit shall be considered properly anchored and spliced where lengths of laps are not less than 1.7 ℓ_d **LRFD [5.11.2.6.4]**. In members not less than 18 inches deep, the length of the stirrup leg for anchoring closed stirrup splices is described in **LRFD [5.11.2.6.4]**.

The Bureau of Structures interprets the lap length to be used for temperature and distribution reinforcement to be a Class "A" splice (using "top" or "others", as appropriate). See Table 9.9-1 and Table 9.9-2 for definition of "top" bars.

The required development length, ℓ_{dh}, for bars in tension terminating in a standard hook is detailed in **LRFD [5.11.2.4]**. This length increases with the bar size. The basic development length, ℓ_{hb}, for a hooked bar is a function of bar diameter, d_b, and concrete strength, f'_c. The basic development length is multiplied by applicable modification factors to produce the required development length, ℓ_{dh}.

Embedment depth is increased for dowel bars (with hooked ends) that run from column or retaining wall into the footing, if the hook does not rest on top of the bar steel mat in the bottom of the footing. This is a construction detail which is the preferred method for anchoring these bars before the concrete is placed.

Dowel bars are used as tensile reinforcement to tie columns or retaining walls to their footings. The amount of bar steel can be reduced by varying the dowel bar lengths projecting above the footing, so that only half the bars are spliced in the same plane. This is a consideration for long retaining walls and for some columns. This allows a Class "B" splice to be used, as opposed to a Class "C" splice where equal length dowel bars are used and all bars are spliced in the same plane.

The length of lap, ℓ_c , for splices in compression is provided in **LRFD [5.11.5.5.1]**.

9.3.2 Bends and Hooks for Deformed Bars

Figure 9.9-1 shows standard hook and bend details for development of longitudinal tension reinforcement. Figure 9.9-2 shows standard hook and bend details for transverse reinforcement (open stirrups and ties). Figure 9.9-3 shows details for transverse reinforcement (closed stirrups). Dimensions for the bending details are shown as out to out of bar, as stated in the *Standard Specifications* Section 505.3.2. The diameter of a bend, measured on the inside of the bar for a standard bend is specified in **LRFD [5.10.2.3]**. Where a larger bend radius is required (non-standard bend) show the inside bend radius on the bar detail. When computing total bar lengths account for the accumulation in length in the bends. Use the figures mentioned above to account for accumulation in length for standard hooks and bends. One leg of bent bars is not dimensioned so that the tolerance for an error in the length due to bending is placed there. Fabrication tolerances for bent bars are specified in the *Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practices* or the *American Concrete Institute (ACI) Detailing Manual* as stated in Section 505.2.1 of the *Standard Specifications*.

Figure 9.3-1 shows typical detailing procedures for bars with bends.

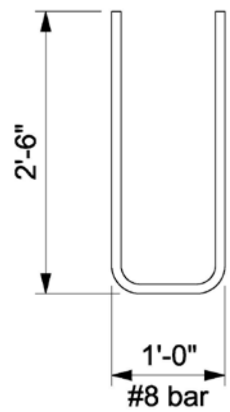


Figure 9.3-1
Bar Bend Detail (#8 bar)

Bar length = 1.0 ft + (2)(2.5 ft) – (2)(0.21 ft) = 5.58 ft or 5'-7" (to the nearest inch)

Where (0.21 ft) is (2.5"/12) and is the standard bar bend deduction found in Figure 9.9-1 for a #8 bar bent 90°.

9.3.3 Bill of Bars

Figure 9.9-4 shows a sample Bill of Bars table for a concrete slab. Different bar letter designations are used for abutments, slabs, and culverts, etc. If bundled bars are used, place a symbol adjacent to the bar mark of the bundled bars and a note below the Bill of Bars table stating the symbol represents bars to be bundled. A column for Bar Series is included if bars are cut.

9.3.4 Bar Series

A Bar Series table enables the detailer to detail bar steel in the simplest manner if it is used properly. Also, it helps the fabricator to prepare the Bill of Bars table.

The following general rules apply to the Bar Series table:

- Equal spacing of bars is required.
- There may be more than 1 Series with same number of bars.
- The total length of a bar is 60 feet (maximum).
- The minimum number of bars per Series is 4.

- Bent bars are bent after cutting.
- Set numbers are assigned to each Series used.

Figure 9.9-5 provides a sample layout for a Bar Series table. The Bill of Bars table will show the number of bars and the average bar length in the Series.

9.4 Steel

Structural steel is used in highway structures throughout Wisconsin. It is used for steel plate I-girders, rolled I-girders and box girders. Steel used for these three superstructure types are typically ASTM A709 Grades 36, 50 and 50W, but may also include high performance steel (HPS). Information on materials used for these superstructure types is provided in 24.2. Other types of steel superstructures are trusses, tied arches and cable-stayed bridges.

Steel is also used in other parts of the structure, such as:

- Bearings (Type A, B, A-T and top/interior plates for Laminated Elastomeric Bearings)
- Piling (H-Piles and CIP-Pile shells)
- Expansion Devices (single strip seal or modular joint)
- Drains (frame, grate and bracket)
- Railings (Type W, H, NY, M, PF, Tubular Screening, Fencing and Combination Railing)
- Steel diaphragms (attached to prestressed girders)

Structural carbon steel (ASTM A709 Grade 36) is used in components that are part of railings, laminated elastomeric bearings and for steel diaphragms attached to prestressed girders. Structural carbon steel (ASTM A36) is used in components that are part of drains. The minimum yield strength is 36 ksi.

High strength structural steel (ASTM A709 Grade 50) is used in H-piles and components that are part of railings and laminated elastomeric bearings. High strength structural weathering steel (ASTM A709 Grade 50W) is used in bearings. The minimum yield strength is 50 ksi.

Structural steel tubing (ASTM A500 Grades B,C) is used in components that are part of railings, such as posts or rail members. The minimum yield strengths will have values around 46 to 50 ksi.

Steel pipe pile material (ASTM A252 Grade 2) is used as the shell to form cast-in-place (CIP) concrete piles. The minimum yield strength is 35 ksi.

Corrugated sheet steel (AASHTO M180, Class A, Type 2) is used as rail members for steel railing Type “W”. The minimum yield strength is 50 ksi.

Stainless steel (ASTM A240 Type 304) can be found as sheets on the surface of top plates for Type A and A-T bearings. It is also used for anchor plates cast into the ends of prestressed girders.

The grade of steel, ASTM Specification (or AASHTO Material Specification) associated with the bulleted items listed above (and their components) can be found in the *Bridge Manual* Chapters or Standards corresponding to these items. This information may also be found in the *Standard Specifications* or “*Special Provisions*”.

The modulus of elasticity of steel, E_s , is 29,000 ksi and the coefficient of thermal expansion is 6.5×10^{-6} in/in/°F per **LRFD [6.4.1]**.

9.5 Miscellaneous Metals

The *Standard Specifications* provide the requirements for other materials made of metal that are used in highway structures. Some metals used or new products containing metal may be covered in the "*Special Provisions*".

Some of these metals, their applications and the Section of the *Standard Specifications* where they are covered are described below.

- Lubricated bronze plates are used on Type A expansion bearings. The requirements for these plates are found in Section 506.2.3.4.

- Bridge name plates are made from a casting of copper, lead, zinc and tin. The requirements for name plates are found in Section 506.2.4.

- Prestressed strands (low relaxation) are made from high tensile strength, 7-wire strands (0.5 or 0.6 inch diameter). The requirements for these strands are found in Section 503.2.3.

- Gray iron castings conforming to ASTM A48, Class 30 are used on Type GC floor drains and downspouts.

- Galvanized standard pipe conforming to ASTM A53 is used for downspouts on Type H floor drains.

- Sheet copper may be used as a waterstop for railroad bridges or as a flashing on movable bridge operator buildings. The requirements for these sheets are found in Section 506.2.3.9.

- Zinc plates may be used at deflection joints in sidewalks and parapets. The requirements for these plates are found in Section 506.2.3.10.

- Shear connectors are welded to the top flanges of steel girders to make the deck composite with the girder. Requirements for these connectors are in Section 506.2.7.

- Aluminum is used for sign bridges and some railings (Tubular Railing Type H). See Section 641.2.7 for sign bridges and Section 513.2.2 for railings that are made from aluminum. For sign bridges and sign supports made from steel, see Section 641.2.8 and 635.2 respectively.

- Steel grid floors are prefabricated grids set on girders and/or stringers. The top of the grid becomes the roadway surface. See Section 515 for the requirements for this steel.

- Welded deformed steel wire fabric has been used as an alternate to stirrup reinforcement for prestressed girders. It shall conform to the requirements of ASTM A1064 as shown on the Chapter 19 – Standards.

9.6 Timber

Timber has been used for timber structures on local roads in Wisconsin. Timber has also been used for piling, railings, falsework, formwork and as backing planks between or behind piling to retain soil.

The *Bridge Manual* and the *Standard Specifications* provide requirements for timber used in highway structures. These locations are highlighted below.

- Timber structures have material requirements that are covered in Chapter 23 of the *Bridge Manual*. Requirements for the condition of the timber members and applicable preservative treatments are covered in Section 507 of the *Standard Specifications*.

- Timber railings for timber structures have material requirements that are covered in Chapter 23 of the *Bridge Manual*. Requirements for the condition of the timber members are covered in Section 507 of the *Standard Specifications*.

- Timber backing plank requirements are covered in 12.10.

9.7 Miscellaneous Materials

Several types of materials are being used as part of a bridge deck protective system. Epoxy coated reinforcing steel, mentioned earlier, is part of this system. Some of these materials or products, are experimental and are placed on specific structures and then monitored and evaluated. A list of materials or products that are part of a bridge deck protective system and are currently used or under evaluation are:

- Galvanized or stainless steel reinforcing bars
- Waterproofing membrane with bituminous concrete overlay
- Microsilica modified concrete or polymer impregnated concrete
- Low slump concrete overlays
- Low-viscosity crack sealer
- Cathodic protection systems with surface overlays

Other materials or products used on highway structures are:

- Downspouts for Type GC and H drains may be fabricated from fiberglass conforming to ASTM D2996, Grade 1, Class A.
- Elastomeric bearing pads (non-laminated) are primarily used with prestressed “I” girders at fixed abutments and piers and at semi-expansion abutments. They are also used with prestressed “slab and box” sections at all supports. The requirements for these pads are described in Section 506.2.6.4 of the *Standard Specifications*.
- Elastomeric bearing pads (laminated) are primarily used with prestressed “I” girders at expansion supports. The requirements for these pads are described in Section 506.2.6.5 of the *Standard Specifications*.
- Preformed fillers are placed vertically in the joint between wing and diaphragm in A1 and A5 abutments, in the joint between wing and barrel in box culverts and at expansion joints in concrete cast-in-place retaining walls. Preformed fillers are placed along the front top surface of A1 and A5 abutments, along the outside top surfaces of fixed piers and under flanges between elastomeric bearing pad (non-laminated) and top edge of support. Cork filler is placed vertically on semi-expansion abutments. The requirements for fillers are described in Section 502.2.7 of the *Standard Specifications*.
- Polyethelene sheets are placed on the top surface of semi-expansion abutments to allow movement of the superstructure across the bearing surface. They are also placed between the structural approach slab and the subgrade, and the approach slab and its footing.

- Rubberized waterproofing membranes are used to seal horizontal and vertical joints at the backface of abutments, culverts and concrete cast-in-place retaining walls. See Section 5.16.2.3 of the *Standard Specifications*.

- Non-staining gray non-bituminous joint sealer is used to seal exposed surfaces of preformed fillers placed in joints as described above. It is also used to place a seal around exposed surfaces of plates used at deflection joints and around railing base plates. The requirement for this joint sealer is referenced in Section 502.2.9 of the *Standard Specifications*.

- Plastic plates may be used at deflection joints in sidewalks and parapets.

- Preformed Fabric, Class A, has been used as a bearing pad under steel bearings. The requirement for this material is given in Section 506.2.6.3 of the *Standard Specifications*.

- Neoprene strip seals are used in single cell and multi-cell (modular) expansion devices.

- Teflon sheets are bonded to steel plates in Type A-T expansion bearings. The requirements for these sheets are found in Section 506.2.8.3 of the *Standard Specifications*.

- Asphalt panels are used on railroad structures to protect the rubber membrane on top of the steel ballast plate from being damaged by the ballast. The requirements for these panels are in the "*Special Provisions*".

- Geotextile fabric is used for drainage filtration, and under riprap and box culverts. This fabric consists of sheets of woven or non-woven synthetic polymers or nylon. Type DF is used for drainage filtration in the pipe underdrain detail placed behind abutments and walls. The fabric allows moisture to drain to the pipe while keeping the backfill from migrating into the coarse material and then into the pipe. Type DF is also used behind abutments or walls that retain soil with backing planks between or behind piling and also for some of the walls detailed in Chapter 14 – Retaining Walls. This fabric will allow moisture to pass through the fabric and the joints in the walls without migration of the soil behind the wall. Type R or HR is placed below riprap and will keep the soil beneath it from being washed away. Type C is placed under breaker run when it is used under box culverts. The requirements for these fabrics are found in Section 645.2 of the *Standard Specifications*.

9.8 Painting

All highway grade separation structures require steel girders to be painted because unpainted steel is subject to additional corrosion from vehicle salt spray. Additional discussion on painting is presented in Chapter 24 – Steel Girder Structures. The current paint system used for I-girders is the three-coat epoxy system specified in Section 517 of the *Standard Specifications*. Tub girders utilize a two-coat polysiloxane system, which includes painting of the inside of the tubs.

Recommended standard colors and paint color numbers for steel girders in Wisconsin in accordance with Federal Standard No. 595C as printed are:

White (For Inside of Box Girders)	#27925
Blue (Medium Sky Blue Tone)	#25240
[1] Brown (Similar to Weathering Steel)	#20059
Gray (Light Gray)	#26293
Green (Medium Tone)	#24260
Reddish-Brown (Red Brick Tone)	#20152
Gray (Dark Gray-DNR Request)	#26132
Black	#27038

Table 9.8-1
Standard Colors for Steel Girders

[1] Shop applied color for weathering steel.

Federal Standard No. 595C can be found at www.federalstandardcolor.com/

All steel bearing components which are not welded to the girder or do not have a Teflon or bronze surface, and anchor bolts shall be galvanized. In addition to galvanizing, some bearing components may also be field or shop painted as noted in the Standards for Chapter 27 – Bearings.

All new structural steel is blast cleaned including weathering steel. It has been shown that paint systems perform well over a longer period of time with proper surface preparation. The blast cleaned surface is a very finely pitted surface with pit depths of 1 ½ mils.

Corrosion of structural steel occurs if the agents necessary to form a corrosion cell are present. A corrosion cell is similar to a battery in that current flows from the anode to the cathode. As the current flows, corrosion occurs at the anode and materials expand. Water carries the electrical current between the anode and cathode. If there is salt in the water, the current travels much faster and the rate of corrosion is accelerated. Oxygen combines with the materials to help form the anodic corrosion cell.

The primary reason for painting steel structures is for the protection of the steel surface. Appearance is a secondary function that is maintained by using compatible top coatings over epoxy systems. Regarding appearance with respect to color retention, black is good, blues and greens are decent, and reddish browns are acceptable, but not the best. Reds are highly discouraged and should not be used.

Paint applied to the steel acts as a moisture barrier. It prevents the water from contacting the steel and then a corrosion cell cannot be formed. When applying a moisture barrier, it is important to get an adhering, uniform thickness as well as an adequate thickness. The film thickness of paint wears with age until it is finally depleted. At this point the steel begins to corrode as moisture is now present in the corrosion cell. If paint is applied too thick, it may crack and/or check due to temperature changes and allow moisture to contact the steel long before the film thickness wears down.

The paint inspector uses a paint gauge to randomly measure the film thickness of the paint according to specifications. Wet film thickness is measured and it is always thicker than the dry film thickness. A vehicle is added to the paint solids so that the solids can be applied to a surface and then it evaporates leaving only the solids on the surface. The percent of solids in a gallon of paint gives an estimate of the wet to dry film thickness ratio.

Refer to Section 1.3.14 of the *Wisconsin Structure Inspection Manual* for the criteria covering spot painting versus complete painting of existing structures. This Section provides information for evaluating the condition of a paint system and recommended maintenance.

Recommended standard colors and color numbers for concrete in Wisconsin in accordance with Federal Standard No. 595C as printed are:

Pearl Gray	#26622
Medium Tan	#33446
Gray Green	#30372
Dark Brown	#30140
Dawn Mist (Grayish Brown)	#36424
Lt. Coffee (Creamy Brown)	#33722

Table 9.8-2
Standard Colors for Concrete

Most paints require concrete to be a minimum of 30 days old before application. This should be considered when specifying completion times for contracts.

9.9 Bar Tables and Figures

(f'c = 3500 psi; fy = 60 ksi)											
BAR SPACING	BAR SIZE		4	5	6	7	8	9	10	11	TYPE
6" OR MORE	CLASS A 1.0 ℓ_d	TOP [1]	1-2 1-5	1-5 1-9	1-9 2-1	2-3 2-9	3-0 3-8	3-9 4-7	4-10 5-10	5-11 7-2	UNCOATED EPOXY
		OTHERS	1-0 1-0	1-0 1-6	1-3 1-10	1-8 2-5	2-2 3-3	2-9 4-1	3-5 5-2	4-3 6-4	UNCOATED EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-6 1-9	1-10 2-3	2-3 2-8	3-0 3-7	3-11 4-8	4-11 5-11	6-3 7-7	7-8 9-3	UNCOATED EPOXY
		OTHERS	1-1 1-3	1-4 2-0	1-7 2-5	2-2 3-2	2-9 4-2	3-6 5-3	4-5 6-8	5-6 8-2	UNCOATED EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	1-11 2-4	2-5 2-11	2-11 3-6	3-10 4-8	5-1 6-2	6-5 7-9	8-1 9-10	10-0 12-1	UNCOATED EPOXY
		OTHERS	1-5 1-8	1-9 2-7	2-1 3-1	2-9 4-2	3-8 5-5	4-7 6-10	5-10 8-8	7-2 10-8	UNCOATED EPOXY
LESS THAN 6"	CLASS A 1.0 ℓ_d	TOP [1]	1-5 1-9	1-9 2-2	2-2 2-7	2-10 3-5	3-9 4-6	4-9 5-9	6-0 7-3	7-4 8-11	UNCOATED EPOXY
		OTHERS	1-0 1-3	1-3 1-11	1-6 2-3	2-1 3-1	2-8 4-0	3-5 5-1	4-3 6-5	5-3 7-10	UNCOATED EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-10 2-3	2-4 2-10	2-9 3-4	3-8 4-6	4-10 5-10	6-1 7-5	7-9 9-5	9-6 11-7	UNCOATED EPOXY
		OTHERS	1-4 1-7	1-8 2-6	2-0 3-0	2-8 3-11	3-6 5-2	4-5 6-7	5-7 8-4	6-10 10-2	UNCOATED EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	2-5 2-11	3-0 3-8	3-7 4-5	4-10 5-10	6-4 7-8	8-0 9-8	10-2 12-4	12-5 15-1	UNCOATED EPOXY
		OTHERS	1-9 2-1	2-2 3-3	2-7 3-10	3-5 5-2	4-6 6-9	5-9 8-7	7-3 10-10	8-11 13-4	UNCOATED EPOXY

Table 9.9-1
Tension Lap Splice Length or Development Length - Deformed Bars
LRFD [5.11.2.1, 5.11.5.3.1]

[1] Top Bar – is a horizontal or nearly horizontal bar with 12 inches of fresh concrete cast below it.

CLASS A - [A_s provided/A_s required] ≥ 2; Bars spliced are 75% or less.

CLASS B - [A_s provided/A_s required] < 2; Bars spliced are 50% or less (or) [A_s provided/A_s required] ≥ 2; Bars spliced are greater than 75%.

CLASS C - [A_s provided/A_s required] < 2; Bars spliced are greater than 50%.

(f'_c = 4000 psi; f_y = 60 ksi)											
BAR SPACING	BAR SIZE		4	5	6	7	8	9	10	11	TYPE
6" OR MORE	CLASS A 1.0 ℓ_d	TOP [1]	1-2 1-5	1-5 1-9	1-9 2-1	2-2 2-7	2-10 3-5	3-6 4-3	4-6 5-5	5-6 6-8	UNCOATED EPOXY
		OTHERS	1-0 1-0	1-0 1-6	1-3 1-10	1-6 2-3	2-0 3-0	2-6 3-9	3-3 4-10	3-11 5-11	UNCOATED EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-6 1-9	1-10 2-3	2-3 2-8	2-9 3-4	3-8 4-5	4-7 5-7	5-10 7-1	7-2 8-8	UNCOATED EPOXY
		OTHERS	1-1 1-3	1-4 2-0	1-7 2-5	2-0 3-0	2-7 3-11	3-3 4-11	4-2 6-3	5-1 7-8	UNCOATED EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	1-11 2-4	2-5 2-11	2-11 3-6	3-7 4-5	4-9 5-9	6-0 7-3	7-7 9-3	9-4 11-4	UNCOATED EPOXY
		OTHERS	1-5 1-8	1-9 2-7	2-1 3-1	2-7 3-10	3-5 5-1	4-3 6-5	5-5 8-2	6-8 10-0	UNCOATED EPOXY
LESS THAN 6"	CLASS A 1.0 ℓ_d	TOP [1]	1-5 1-9	1-9 2-2	2-2 2-7	2-8 3-3	3-6 4-3	4-5 5-4	5-7 6-9	6-10 8-4	UNCOATED EPOXY
		OTHERS	1-0 1-3	1-3 1-11	1-6 2-3	1-11 2-10	2-6 3-9	3-2 4-9	4-0 6-0	4-11 7-4	UNCOATED EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-10 2-3	2-4 2-10	2-9 3-4	3-5 4-2	4-6 5-6	5-9 6-11	7-3 8-10	8-11 10-10	UNCOATED EPOXY
		OTHERS	1-4 1-7	1-8 2-6	2-0 3-0	2-6 3-8	3-3 4-10	4-1 6-2	5-2 7-9	6-5 9-7	UNCOATED EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	2-5 2-11	3-0 3-8	3-7 4-5	4-6 5-6	5-11 7-2	7-6 9-1	9-6 11-6	11-8 14-2	UNCOATED EPOXY
		OTHERS	1-9 2-1	2-2 3-3	2-7 3-10	3-3 4-10	4-3 6-4	5-4 8-0	6-9 10-2	8-4 12-6	UNCOATED EPOXY

<u>Table 9.9-2</u>

Tension Lap Splice Length or Development Length – Deformed Bars

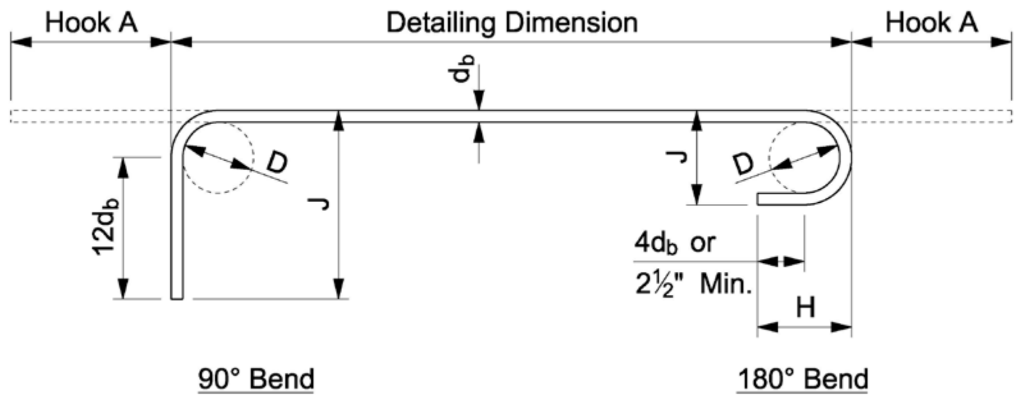
LRFD [5.11.2.1, 5.11.5.3.1]

[1] Top Bar – is a horizontal or nearly horizontal bar with 12 inches of fresh concrete cast below it.

CLASS A – [A_s provided/A_s required] ≥ 2; Bars spliced are 75% or less.

CLASS B – [A_s provided/A_s required] < 2; Bars spliced are 50% or less (or) [A_s provided/A_s required] ≥ 2; Bars spliced are greater than 75%.

CLASS C - [A_s provided/A_s required] < 2; Bars spliced are greater than 50%.



d_b = nominal diameter of reinforcing bar (in)

Definition of standard hooks **LRFD [5.10.2.1, C5.11.2.4.1]**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3]**

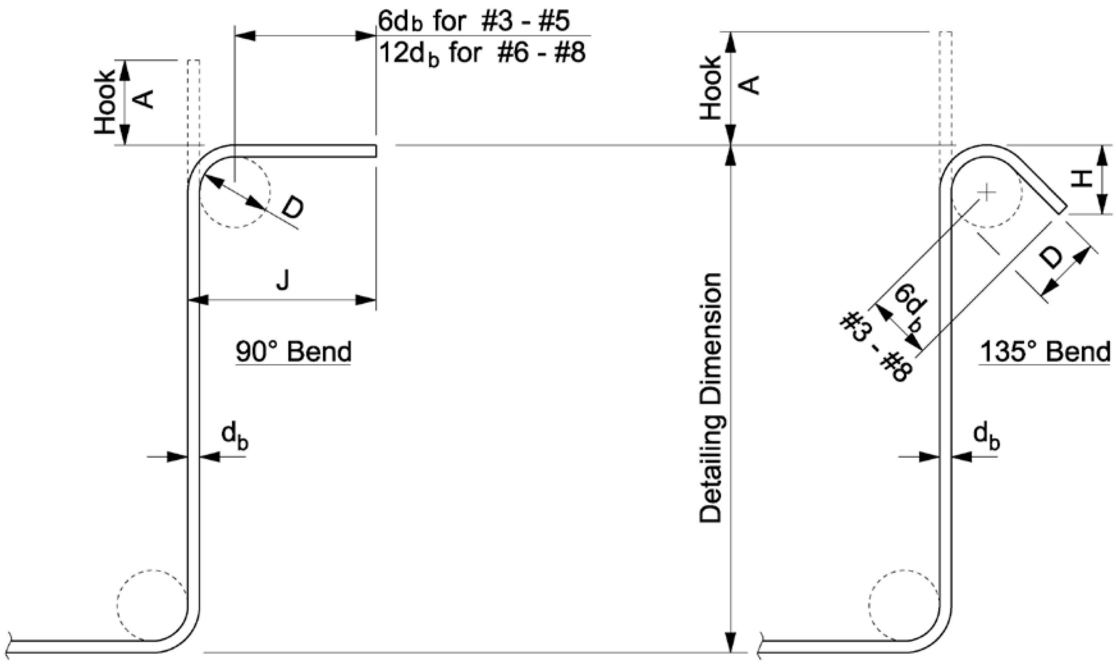
D = $6d_b$ FOR #3 THRU #8

D = $8d_b$ FOR #9, #10, and #11

BAR SIZE	MINIMUM HOOK, ALL GRADES					
	90° HOOKS			180° HOOKS		
	HOOK A	J	J MINUS HOOK A [1]	HOOK A	J	APPROX. H
4	7	8	1	6	4	4 ½
5	8 ½	10	1 ½	7	5	5
6	10	1-0	2	8	6	6
7	1-0	1-2	2	10	7	7
8	1-1 ½	1-4	2 ½	11	8	8
9	1-4	1-7	3	1-3	11 ¼	10 ¼
10	1-6	1-9 ½	3 ½	1-5	1-0 ¾	11 ½
11	1-8	2-0	4	1-7	1-2 ¼	1-0 ¾

Figure 9.9-1
Standard Hooks and Bends for Deformed Longitudinal Reinforcement

[1] "J" MINUS "HOOK A" = DEDUCTION FOR ONE BEND



d_b = nominal diameter of reinforcing bar (in)

Definition of Standard Hooks **LRFD [5.10.2.1]**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3]**

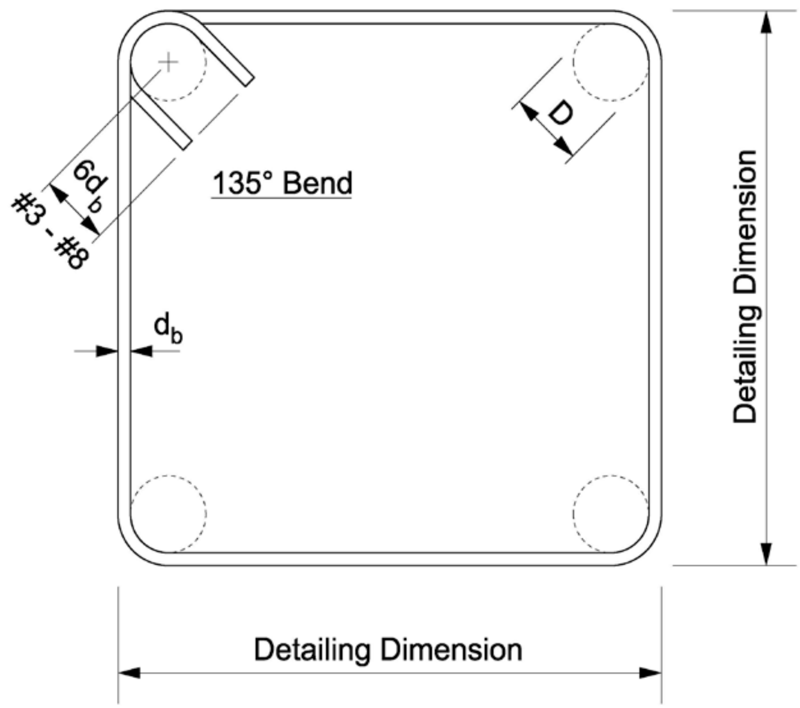
D = $4d_b$ FOR #3 THRU #5

D = $6d_b$ FOR #6 THRU #8

MINIMUM HOOK, ALL GRADES					
		90° HOOKS		135° HOOKS	
BAR SIZE	D	HOOK A	APPROX J	HOOK A	H
3	1 ½	3	4	4	2 ½
4	2	3 ½	4 ½	4 ½	3
5	2 ½	4 ½	6	5 ½	3 ¾
6	4 ½	10	1-0	8	4 ½

Figure 9.9-2

Standard Hooks and Bends for Deformed Transverse Reinforcement
(Stirrups and Ties)



Stirrup Bar Length equals sum of all Detailing Dimensions plus "Stirrup Add-On" from table

d_b = nominal diameter of reinforcing bar (in)

Definition of Standard Hooks **LRFD [5.10.2.1]**

MINIMUM BEND DIAMETER (D) – **LRFD [5.10.2.3]**

D = $4d_b$ FOR #3 THRU #5

D = $6d_b$ FOR #6 THRU #8

BAR SIZE	D	STIRRUP ADD-ON
3	1 ½	5
4	2	6
5	2 ½	8
6	4 ½	10
7	5 ¼	12
8	6	13

Figure 9.9-3
Standard Details and Bends for Deformed Transverse Reinforcement
(Closed Stirrups)

BILL OF BARS

NOTE: THE FIRST OR FIRST TWO DIGITS OF THE BAR MARK SIGNIFIES THE BAR SIZE.

BAR MARK	COAT	NO. REQ'D	LENGTH	BENT	BAR SERIES	LOCATION
S501		10	4-2		∆	SLAB - TRANS.
S502		20	6-3		∆	SLAB - TRANS.
S503	X	19	42-8			SLAB - LONG.

∆ LENGTH SHOWN FOR BAR IS AN AVERAGE LENGTH AND SHOULD ONLY BE USED FOR BAR WEIGHT CALCULATIONS. SEE BAR SERIES TABLE FOR ACTUAL LENGTHS.

Figure 9.9-4
Bill of Bars

BAR SERIES TABLE

MARK	NO. REQ'D.	LENGTH
S501	1 SERIES OF 10	2-1 TO 6-3
S502	2 SERIES OF 10	3-2 TO 9-5

BUNDLE AND TAG EACH SERIES SEPARATELY

Figure 9.9-5
Bar Series Table

BAR SIZE	BAR WEIGHT (lbs/ft)	NOM. DIA (in)	NOM. AREA (in²)	NUMBER OF BARS								
				2	3	4	5	6	7	8	9	10
4	0.668	0.500	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60	1.80	2.00
5	1.043	0.625	0.31	0.62	0.93	1.24	1.55	1.86	2.17	2.48	2.79	3.10
6	1.502	0.750	0.44	0.88	1.32	1.76	2.20	2.64	3.08	3.52	3.96	4.40
7	2.044	0.875	0.60	1.20	1.80	2.40	3.00	3.60	4.20	4.80	5.40	6.00
8	2.670	1.000	0.79	1.58	2.37	3.16	3.95	4.74	5.53	6.32	7.11	7.90
9	3.400	1.128	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00
10	4.303	1.270	1.27	2.54	3.81	5.08	6.35	7.62	8.89	10.16	11.43	12.70
11	5.313	1.410	1.56	3.12	4.68	6.24	7.80	9.36	10.92	12.48	14.04	15.60

Table 9.9-3
Bar Areas Per Number of Bars (in^2)

BAR SIZE	4 ½“	5”	5 ½“	6”	6 ½“	7”	7 ½”	8”	8 ½”	9”	10”	11”	12”
4	0.52	0.47	0.43	0.39	0.36	0.34	0.31	0.29	0.28	0.26	0.24	0.21	0.20
5	0.82	0.74	0.67	0.61	0.57	0.53	0.49	0.46	0.43	0.41	0.37	0.33	0.31
6	1.18	1.06	0.96	0.88	0.82	0.76	0.71	0.66	0.62	0.59	0.53	0.48	0.44
7	1.60	1.44	1.31	1.20	1.11	1.03	0.96	0.90	0.85	0.80	0.72	0.66	0.60
8	2.09	1.88	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	0.94	0.86	0.79
9	---	2.40	2.18	2.00	1.85	1.71	1.60	1.50	1.41	1.33	1.20	1.09	1.00
10	---	3.04	2.76	2.53	2.34	2.17	2.02	1.90	1.79	1.69	1.52	1.38	1.27
11	---	3.75	3.41	3.12	2.88	2.68	2.50	2.34	2.21	2.08	1.87	1.70	1.56

Table 9.9-4
Area of Bar Reinf. (in^2 / ft) vs. Spacing of Bars (in)

9.10 Granular Materials

Several types of granular materials are used for backfilling excavations, providing foundation improvements, and reinforcing soils. Table 9.10-5 provides recommended uses and notes for commonly used granular materials for structures. Refer to the specifications for material gradations, testing, compaction, and other requirements specific for the application. Refer to 6.4.2 for plan preparations.

Granular pay limits should be shown on all structure plans. See Standards for typical backfill limits and plan notes.

<table>
<tr><th>Granular Material Type</th><th>Uses</th><th>Notes</th></tr>
<tr><td>Backfill Structure – Type A</td><td><u>Backfill</u>
• Abutments
• Retaining walls</td><td rowspan="2">Type A facilitates better drainage than Type B.

Type A may be substituted for Type B material per specifications.</td></tr>
<tr><td>Backfill Structure – Type B</td><td><u>Backfill</u>
• Box culverts
• Structural plate pipes
• Pipe arches
<u>Retained Backfill (if needed)</u>
▪ Various structures
<u>Foundation</u>
• Abutments
• Retaining walls</td></tr>
<tr><td>Backfill Granular – Grade 1</td><td rowspan="2">Refer to FDM for usages</td><td rowspan="2">Grade 1 may be substituted for Grade 2 material per specifications.</td></tr>
<tr><td>Backfill Granular – Grade 2</td></tr>
<tr><td>Base Aggregate Dense 1 1/4-inch</td><td>• Structural approach (base)
• GRS Walls (reinforced soil foundation and approach)</td><td></td></tr>
<tr><td>Reinforced Soils</td><td>• MSE Walls</td><td>Backfill included in MSE Wall bid items.</td></tr>
<tr><td>Base Aggregate Open Graded</td><td>• GRS Walls (reinforced soil)
• MSE Walls (for elevations below HW100)</td><td></td></tr>
<tr><td>Breaker Run</td><td>• Box culverts (foundation)</td><td>See Standard Detail 9.01 for alternatives and notes</td></tr>
<tr><td>Flowable Backfill</td><td>• Soldier pile walls</td><td></td></tr>
</table>

Table 9.10-5

Recommendations for Granular Material Usage

9.11 References

1. Ghorbanpoor, A., Kriha, B., Reshadi, R. *Aesthetic Coating for Steel Bridge Components – Amended Study*. S.1.: Wisconsin Department of Transportation, Final Report No. 0092-11-07, 2015.

9.12 Appendix - Draft Bar Tables

The following Draft Bar Tables are provided for information only. We expect the tables to be moved into the main text of Chapter 9 in July of 2018, and at that time to begin their use. We are delaying their use to allow time for modification of details and software that are affected.

The 2015 Interim Revisions to the AASHTO LRFD Bridge Design Specifications (7th Edition), modified the tension development lengths and tension lap lengths for straight deformed bars as follows:

The tension development length, ℓ_d ,shall not be less than the product of the basic tension development length, ℓ_{db} , and the appropriate modification factors, λ_i . **LRFD [5.11.2.1.1]**

$$\ell_d = \ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er}) / \lambda$$

in which: $\ell_{db} = 2.4 \cdot d_b \cdot [f_y / (f'_c)^{1/2}]$

where:

ℓ_{db} = basic development length (in.)

λ_{rl} = reinforcement location factor

λ_{cf} = coating factor

λ = conc. density modification factor ; for normal weight conc. = 1.0 , **LRFD [5.4.2.8]**

λ_{rc} = reinforcement confinement factor

λ_{er} = excess reinforcement factor

f_y = specified yield strength of reinforcing bars (ksi)

d_b = diameter of bar (in.)

f'_c = specified compressive strength of concrete (ksi)

Top bars will continue to refer to horizontal bars placed with more than 12" of fresh concrete cast below it. Bars not meeting this criteria will be referred to as Others.

Per **LRFD [5.11.5.3.1]**, there are two lap splice classes, Class A and Class B.

- Class A lap splice ……………………1.0 ℓ_d
- Class B lap splice …………………... 1.3 ℓ_d

The criteria for where to apply each Class is covered in the above reference.

Draft Table

Epoxy Coated (f'_c = 4,000 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-6"	1'-6"	1'-11"	1'-11"
	2.0"	1'-6"	1'-6"	1'-11"	1'-11"
	≥ 2.5"	1'-6"	1'-6"	1'-11"	1'-11"
5	1.5"	2'-3"	2'-3"	3'-0"	3'-0"
	2.0"	2'-3"	1'-10"	3'-0"	2'-4"
	≥ 2.5"	2'-3"	1'-10"	3'-0"	2'-4"
6	1.5"	2'-11"	2'-9"	3'-10"	3'-7"
	2.0"	2'-11"	2'-9"	3'-10"	3'-7"
	≥ 2.5"	2'-11"	2'-2"	3'-10"	2'-10"
7	1.5"	3'-7"	3'-7"	4'-8"	4'-8"
	2.0"	3'-6"	3'-2"	4'-6"	4'-2"
	≥ 2.5"	3'-6"	3'-2"	4'-6"	4'-2"
8	1.5"	4'-6"	4'-6"	5'-11"	5'-11"
	2.0"	4'-6"	3'-8"	5'-11"	4'-9"
	≥ 2.5"	4'-6"	3'-8"	5'-11"	4'-9"
9	1.5"	5'-9"	5'-7"	7'-5"	7'-4"
	2.0"	5'-9"	4'-6"	7'-5"	5'-10"
	≥ 2.5"	5'-9"	4'-1"	7'-5"	5'-4"
10	1.5"	7'-4"	6'-9"	9'-7"	8'-10"
	2.0"	7'-4"	5'-6"	9'-7"	7'-2"
	≥ 2.5"	7'-4"	4'-10"	9'-7"	6'-3"
11	1.5"	9'-1"	8'-2"	11'-9"	10'-7"
	2.0"	9'-1"	6'-8"	11'-9"	8'-7"
	≥ 2.5"	9'-1"	6'-0"	11'-9"	7'-9"

Epoxy Coated (f'_c = 4,000 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-11"	1'-11"	2'-6"	2'-6"
	2.0"	1'-11"	1'-11"	2'-6"	2'-6"
	≥ 2.5"	1'-11"	1'-11"	2'-6"	2'-6"
5	1.5"	2'-7"	2'-7"	3'-4"	3'-4"
	2.0"	2'-7"	2'-4"	3'-4"	3'-1"
	≥ 2.5"	2'-7"	2'-4"	3'-4"	3'-1"
6	1.5"	3'-4"	3'-1"	4'-4"	4'-0"
	2.0"	3'-4"	3'-1"	4'-4"	4'-0"
	≥ 2.5"	3'-4"	2'-10"	4'-4"	3'-8"
7	1.5"	4'-1"	4'-1"	5'-3"	5'-3"
	2.0"	4'-0"	3'-7"	5'-2"	4'-8"
	≥ 2.5"	4'-0"	3'-7"	5'-2"	4'-8
8	1.5"	5'-2"	5'-2"	6'-8"	6'-8"
	2.0"	5'-2"	4'-1"	6'-8"	5'-4"
	≥ 2.5"	5'-2"	4'-1"	6'-8"	5'-4"
9	1.5"	6'-6"	6'-4"	8'-5"	8'-3"
	2.0"	6'-6"	5'-1"	8'-5"	6'-7"
	≥ 2.5"	6'-6"	4'-8"	8'-5"	6'-0"
10	1.5"	8'-4"	7'-8"	10'-10"	10'-0"
	2.0"	8'-4"	6'-3"	10'-10"	8'-1"
	≥ 2.5"	8'-4"	5'-6"	10'-10"	7'-1"
11	1.5"	10'-3"	9'-3"	13'-4"	12'-0"
	2.0"	10'-3"	7'-6"	13'-4"	9'-9"
	≥ 2.5"	10'-3"	6'-10"	13'-4"	8'-10"

Draft Table

Uncoated (f'_c = 4,000 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-3"	1'-3"	1'-7"	1'-7"
	2.0"	1'-3"	1'-3"	1'-7"	1'-7"
	≥ 2.5"	1'-3"	1'-3"	1'-7"	1'-7"
5	1.5"	1'-6"	1'-6"	2'-0"	2'-0"
	2.0"	1'-6"	1'-6"	2'-0"	2'-0"
	≥ 2.5"	1'-6"	1'-6"	2'-0"	2'-0"
6	1.5"	2'-0"	1'-10"	2'-7"	2'-4"
	2.0"	2'-0"	1'-10"	2'-7"	2'-4"
	≥ 2.5"	2'-0"	1'-10"	2'-7"	2'-4"
7	1.5"	2'-5"	2'-5"	3'-1"	3'-1"
	2.0"	2'-4"	2'-2"	3'-0"	2'-9"
	≥ 2.5"	2'-4"	2'-2"	3'-0"	2'-9"
8	1.5"	3'-0"	3'-0	3'-11"	3'-11"
	2.0"	3'-0"	2'-5"	3'-11"	3'-2"
	≥ 2.5"	3'-0"	2'-5"	3'-11"	3'-2"
9	1.5"	3'-10"	3'-9"	5'-0"	4'-10"
	2.0"	3'-10"	3'-0"	5'-0"	3'-11"
	≥ 2.5"	3'-10"	2'-9"	5'-0"	3'-7"
10	1.5"	4'-11"	4'-6"	6'-4"	5'-11"
	2.0"	4'-11"	3'-8"	6'-4"	4'-9"
	≥ 2.5"	4'-11"	3'-3"	6'-4"	4'-2"
11	1.5"	6'-0"	5'-5"	7'-10"	7'-1"
	2.0"	6'-0"	4'-5"	7'-10"	5'-9"
	≥ 2.5"	6'-0"	4'-0"	7'-10"	5'-2"

Uncoated (f'_c = 4,000 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-7"	1'-7"	2'-1"	2'-1"
	2.0"	1'-7"	1'-7"	2'-1"	2'-1"
	≥ 2.5"	1'-7"	1'-7"	2'-1"	2'-1"
5	1.5"	2'-0"	2'-0"	2'-7"	2'-7"
	2.0"	2'-0"	2'-0"	2'-7"	2'-7"
	≥ 2.5"	2'-0"	2'-0"	2'-7"	2'-7"
6	1.5"	2'-7"	2'-4"	3'-4"	3'-1"
	2.0"	2'-7"	2'-4"	3'-4"	3'-1"
	≥ 2.5"	2'-7"	2'-4"	3'-4"	3'-1"
7	1.5"	3'-1"	3'-1"	4'-0"	4'-0"
	2.0"	3'-0"	2'-9"	3'-11"	3'-7"
	≥ 2.5"	3'-0"	2'-9"	3'-11"	3'-7"
8	1.5"	3'-11"	3'-11"	5'-1"	5'-1"
	2.0"	3'-11"	3'-2"	5'-1"	4'-1"
	≥ 2.5"	3'-11"	3'-2"	5'-1"	4'-1"
9	1.5"	5'-0"	4'-10"	6'-5"	6'-4"
	2.0"	5'-0"	3'-11"	6'-5"	5'-1"
	≥ 2.5"	5'-0"	3'-7"	6'-5"	4'-7"
10	1.5"	6'-4"	5'-11"	8'-3"	7'-8"
	2.0"	6'-4"	4'-9"	8'-3"	6'-3"
	≥ 2.5"	6'-4"	4'-2"	8'-3"	5'-5"
11	1.5"	7'-10"	7'-1"	10'-2"	9'-2"
	2.0"	7'-10"	5'-9"	10'-2"	7'-6"
	≥ 2.5"	7'-10"	5'-2"	10'-2"	6'-9"

Draft Table

Epoxy Coated (f'$_c$ = 3,500 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-7"	1'-7"	2'-0"	2'-0"
	2.0"	1'-7"	1'-7"	2'-0"	2'-0"
	≥ 2.5"	1'-7"	1'-7"	2'-0"	2'-0"
5	1.5"	2'-5"	2'-5"	3'-2"	3'-2"
	2.0"	2'-5"	1'-11"	3'-2"	2'-6"
	≥ 2.5"	2'-5"	1'-11"	3'-2"	2'-6"
6	1.5"	3'-2"	2'-11"	4'-1"	3'-9"
	2.0"	3'-2"	2'-11"	4'-1"	3'-9"
	≥ 2.5"	3'-2"	2'-4"	4'-1"	3'-0"
7	1.5"	3'-10"	3'-10"	5'-0"	5'-0"
	2.0"	3'-9"	3'-5"	4'-10"	4'-5"
	≥ 2.5"	3'-9"	3'-5"	4'-10"	4'-5"
8	1.5"	4'-10"	4'-10"	6'-3"	6'-3"
	2.0"	4'-10"	3'-11"	6'-3"	5'-0"
	≥ 2.5"	4'-10"	3'-11"	6'-3"	5'-0"
9	1.5"	6'-1"	6'-0"	7'-11"	7'-10"
	2.0"	6'-1"	4'-10"	7'-11"	6'-3"
	≥ 2.5"	6'-1"	4'-4"	7'-11"	5'-8"
10	1.5"	7'-10"	7'-3"	10'-2"	9'-5"
	2.0"	7'-10"	5'-11"	10'-2"	7'-8"
	≥ 2.5"	7'-10"	5'-2"	10'-2"	6'-8"
11	1.5"	9'-8"	8'-9"	12'-7"	11'-4"
	2.0"	9'-8"	7'-1"	12'-7"	9'-2"
	≥ 2.5"	9'-8"	6'-5"	12'-7"	8'-4"

Epoxy Coated (f'$_c$ = 3,500 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	2'-0"	2'-0"	2'-8"	2'-8"
	2.0"	2'-0"	2'-0"	2'-8"	2'-8"
	≥ 2.5"	2'-0"	2'-0"	2'-8"	2'-8"
5	1.5"	2'-9"	2'-9"	3'-7"	3'-7"
	2.0"	2'-9"	2'-6"	3'-7"	3'-3"
	≥ 2.5"	2'-9"	2'-6"	3'-7"	3'-3"
6	1.5"	3'-7"	3'-4"	4'-7"	4'-3"
	2.0"	3'-7"	3'-4"	4'-7"	4'-3"
	≥ 2.5"	3'-7"	3'-0"	4'-7"	3'-11"
7	1.5"	4'-4"	4'-4"	5'-7"	5'-7"
	2.0"	4'-3"	3'-10"	5'-6"	5'-0"
	≥ 2.5"	4'-3"	3'-10"	5'-6"	5'-0"
8	1.5"	5'-6"	5'-6"	7'-1"	7'-1"
	2.0"	5'-6"	4'-5"	7'-1"	5'-8"
	≥ 2.5"	5'-6"	4'-5"	7'-1"	5'-8"
9	1.5"	6'-11"	6'-10"	9'-0"	8'-10"
	2.0"	6'-11"	5'-5"	9'-0"	7'-1"
	≥ 2.5"	6'-11"	4'-11"	9'-0"	6'-5"
10	1.5"	8'-11"	8'-2"	11'-7"	10'-8"
	2.0"	8'-11"	6'-8"	11'-7"	8'-8"
	≥ 2.5"	8'-11"	5'-10"	11'-7"	7'-7"
11	1.5"	10'-11"	9'-10"	14'-3"	12'-10"
	2.0"	10'-11"	8'-0"	14'-3"	10'-5"
	≥ 2.5"	10'-11"	7'-3"	14'-3"	9'-5"

Draft Table

Uncoated (f'_c = 3,500 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-4"	1'-4"	1'-8"	1'-8"
	2.0"	1'-4"	1'-4"	1'-8"	1'-8"
	≥ 2.5"	1'-4"	1'-4"	1'-8"	1'-8"
5	1.5"	1'-8"	1'-8"	2'-1"	2'-1"
	2.0"	1'-8"	1'-8"	2'-1"	2'-1"
	≥ 2.5"	1'-8"	1'-8"	2'-1"	2'-1"
6	1.5"	2'-1"	1'-11"	2'-9"	2'-6"
	2.0"	2'-1"	1'-11"	2'-9"	2'-6"
	≥ 2.5"	2'-1"	1'-11"	2'-9"	2'-6"
7	1.5"	2'-7"	2'-7"	3'-4"	3'-4"
	2.0"	2'-6"	2'-3"	3'-3"	2'-11"
	≥ 2.5"	2'-6"	2'-3"	3'-3"	2'-11"
8	1.5"	3'-3"	3'-3	4'-2"	4'-2"
	2.0"	3'-3"	2'-7"	4'-2"	3'-4"
	≥ 2.5"	3'-3"	2'-7"	4'-2"	3'-4"
9	1.5"	4'-1"	4'-0"	5'-4"	5'-2"
	2.0"	4'-1"	3'-3"	5'-4"	4'-2"
	≥ 2.5"	4'-1"	2'-11"	5'-4"	3'-10"
10	1.5"	5'-3"	4'-10"	6'-10"	6'-3"
	2.0"	5'-3"	3'-11"	6'-10"	5'-1"
	≥ 2.5"	5'-3"	3'-5"	6'-10"	4'-6"
11	1.5"	6'-5"	5'-10"	8'-5"	7'-7"
	2.0"	6'-5"	4'-9"	8'-5"	6'-2"
	≥ 2.5"	6'-5"	4'-3"	8'-5"	5'-7"

Uncoated (f'_c = 3,500 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-8"	1'-8"	2'-2"	2'-2"
	2.0"	1'-8"	1'-8"	2'-2"	2'-2"
	≥ 2.5"	1'-8"	1'-8"	2'-2"	2'-2"
5	1.5"	2'-1"	2'-1"	2'-9"	2'-9"
	2.0"	2'-1"	2'-1"	2'-9"	2'-9"
	≥ 2.5"	2'-1"	2'-1"	2'-9"	2'-9"
6	1.5"	2'-9"	2'-6"	3'-6"	3'-3"
	2.0"	2'-9"	2'-6"	3'-6"	3'-3"
	≥ 2.5"	2'-9"	2'-6"	3'-6"	3'-3"
7	1.5"	3'-4"	3'-4"	4'-4"	4'-4"
	2.0"	3'-3"	2'-11"	4'-2"	3'-10"
	≥ 2.5"	3'-3"	2'-11"	4'-2"	3'-10"
8	1.5"	4'-2"	4'-2"	5'-5"	5'-5"
	2.0"	4'-2"	3'-4"	5'-5"	4'-4"
	≥ 2.5"	4'-2"	3'-4"	5'-5"	4'-4"
9	1.5"	5'-4"	5'-2"	6'-11"	6'-9"
	2.0"	5'-4"	4'-2"	6'-11"	5'-5"
	≥ 2.5"	5'-4"	3'-10"	6'-11"	4'-11"
10	1.5"	6'-10"	6'-3"	8'-10"	8'-2"
	2.0"	6'-10"	5'-1"	8'-10"	6'-8"
	≥ 2.5"	6'-10"	4'-6"	8'-10"	5'-10"
11	1.5"	8'-5"	7'-7"	10'-11"	9'-10"
	2.0"	8'-5"	6'-2"	10'-11"	8'-0"
	≥ 2.5"	8'-5"	5'-7"	10'-11"	7'-3"

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10.1 General

The purpose of the Geotechnical Investigation is to provide subsurface information for the plans and to develop recommendations for the construction of the structure at reasonable costs versus short and long term performance. The level of Geotechnical Investigation is a function of the type of the structure and the associated performance. For example, a box culvert under a low ADT roadway compared to a multi-span bridge on a major interstate would require a different level of Geotechnical Investigation. The challenge for the geotechnical engineer is to gather subsurface information that will allow for a reasonable assessment of the soil and rock properties compared to the cost of the investigation.

The geotechnical engineer and the structure engineer need to work collectively when evaluating the loads on the structures and the resistance of the soil and rock. The development of the geotechnical investigation and evaluation of the subsurface information requires a degree of engineering judgment. A guide for performing the Geotechnical Investigation is provided in WisDOT Geotechnical Bulletin No. 1, **LRFD [10.4]** and Geotechnical Engineering Circular #5 – Evaluation of Soil and Rock Properties (Sabatini, 2002).

The following structures will require a Geotechnical Investigation:

- Bridges
- Box Culverts
- Retaining Walls
- Non-Standard Sign Structures Foundations
- High Mast Lighting Foundations
- Noise Wall Foundations

10.2 Subsurface Exploration

The Geotechnical Engineering Unit (or geotechnical consultant) prepares the Site Investigation Report (SIR) and the Subsurface Exploration (SE) sheet. The SIR describes the subsurface investigation, laboratory testing, analyses, computations and recommendations for the structure. All data relative to the underground conditions which may affect the design of the proposed structure's foundation are reported. Further information describing this required investigation can be found in the Department's "Geotechnical Bulletin #1" document. The Subsurface Exploration sheet is a CADDS drawing that illustrates the soil boring locations and is a graphical representation of the driller's findings. This sheet is included in the structure plans. If the Department is not completing the geotechnical work on the project, the SIR and SE sheet(s) are the responsibility of the consultant.

The subsurface investigation is composed of two areas of investigation: the Surface Survey and the detailed Site Investigation.

Surface Surveys include studies of the site geology and air-photo review, and they can include geophysical methods of exploration. This work should include a review of any existing structure foundations and any existing geotechnical information. Surface Surveys provide valuable data indicating approximate soil conditions during the reconnaissance phase.

Based on the results of the Surface Survey information, the plans for a Detailed Site Investigation are made. The subsurface investigation needs to provide the following information:

- Depth, extent and thickness of each soil or rock stratum
- Soil texture, color, mottling and moisture content
- Rock type, color and condition
- In-situ field tests to determine soil and rock parameters
- Laboratory samples for determining soil or rock parameters
- Water levels, water loss during drilling, utilities and any other relevant information

The number and spacing of borings is controlled by the characteristics and sequence of subsurface strata and by the size and type of the proposed structure. Depending upon the timing of the Geotechnical Investigation the required information may not be available and the geotechnical engineer may have to develop a subsurface investigation plan based on the initial design. The Department understands that additional investigation may be required once the preliminary design is completed. The challenge for the Department and the consultant is to develop a geotechnical investigation budget without knowing the subsurface conditions that will be encountered. Existing subsurface information from previous work can help this situation, but the plans should be flexible to allow for some unforeseen subsurface conditions.

One particular subsurface condition is the presence of shallow rock. In some cases, borings should be made at a frequency of one per substructure unit to adequately define the subsurface conditions. However, with shallow rock two or more borings may be necessary to define the rock line below the foundation. Alternatively, where it is apparent the soil is uniform, fewer borings are needed. For example, a four span bridge with short (less than30 foot) spans at each end of a bridge may only require three borings versus the five borings (one per substructure).

Borings are typically advanced to a depth where the added stress due to the applied load is 10 percent of the existing stress due to overburden or extended beyond the expected pile penetration depths. Where rock is encountered, borings are advanced by diamond bit coring according to ASTM D2113 to determine rock quality according to ASTM D6032.

LRFD [Table 10.4.2-1] Minimum Number of Exploration Points and Depth of Exploration (modified after Sabatini et al., 2002) provides guidelines for an investigation of bridges (shallow foundations and deep foundations) and retaining walls. The following presents the typical subsurface investigation guidelines for the other structures:

- Box Culverts: The recommended spacing of the borings would be 1/every 200 feet of length of the box culvert with a minimum of two boring for a new box culvert. The borings should have 15 feet of continuous SPT samples below the base of the box culvert.

- Box Culvert Extensions: May require a boring depending upon the length of the extension and the available information from the existing box culvert. If a boring is recommended then it would follow the same procedures as for a new box culvert.

- Non-Standard Sign Structure Foundations: The recommended spacing would be one for each sign structure site. If the sign structure is a bridge with two foundations then one boring may still be adequate. The borings should have 20 feet of continuous SPT samples and a SPT sample at 25 feet and 30 feet below the ground surface at the sign structure site.

- High Mast Lighting Foundations: The recommended spacing would be one for each site. The borings should have 15 feet of continuous SPT samples and a SPT sample every 5 feet to a depth of 40 feet below the ground surface at the site.

- Noise Wall Foundations: The recommended spacing would be one for every 200 feet to 300 feet of wall. The borings should have 20 feet of continuous SPT samples below the ground surface.

The Department generally follows AASHTO laboratory testing procedures. Any or all of the following soil tests may be considered necessary or desirable at a given site:

In-situ (field) Tests

- Standard penetration
- Pocket penetrometer (cohesive soil)
- Vane shear (cohesive soil)
- Cone penetration (seldom used)
- Rock core recovery and Rock Quality Designation (RQD)

Laboratory Tests

- Moisture, density, consistency limits and unit weight
- Unconfined compression (cohesive soils and rock cores)
- Grain size analysis (water crossings) - This test is required for streambed sediments of multi-span structures over water to facilitate scour computations.
- One-dimensional consolidation (seldom used)
- Unconsolidated undrained triaxial compression (seldom used)
- Consolidated undrained triaxial compression with pore water pressure readings (seldom used)
- Corrosion Tests (pH, resistivity, sulfate, chloride and organic content)

One of the most widely used in-situ tests in the United States is the Standard Penetration Test (AASHTO T-206) as described in the *AASHTO Standard Specifications*. This test provides an indication of the relative density of cohesionless soils and, along with the pocket penetrometer readings, predicts the consistency and undrained shear strength of cohesive soils. Standard Penetration Tests (SPTs) generally consist of driving a 2-inch O.D. split barrel sampler into the ground with a 140-pound hammer falling over a height of 30 inches. The split-barrel sampler is driven in 6-inch increments for a total of 18-inches and the number of blows for each 6-inch increment is recorded. The field blow-count, SPT N-value, equals the number of blows that are required to drive the sampler the last 12-inches of penetration. Split-barrel samplers are typically driven with a conventional donut, safety or automatic-trip hammer. Hammer efficiencies, ER, are determined in accordance with ASTM D 4945. In lieu of a more detailed assessment, ER values of 45, 60 and 80 percent may be used to compute corrected blow counts, N_{60}, for conventional, safety and automatic-trip hammers, respectively, in accordance with **LRFD [10.4.6.2.4]**. Correlation between standard penetration values and the resulting soil bearing value approximations are available from many sources. Standard penetration values can be used by experienced Geotechnical Engineers to estimate pile shaft resistance values by also considering soil texture, moisture content, location of water table, depth below proposed footing and method of boring advance.

For example, DOT Geotechnical Engineers using DOT soil test information know that certain sand and clays in the northeastern part of Wisconsin have higher load-carrying capacities than tests indicate. This information is confirmed by comparing test pile data at the different sites to computed values. The increased capacities are realized by increasing the design point resistance and/or shaft resistance values in the Site Investigation Report.

Wisconsin currently uses most of the soil tests previously mentioned. The soil tests used for a given site are determined by the complexity of the site, size of the project and availability of funds for subsurface investigation. The scope and extent of the laboratory testing program should take into consideration available subsurface information obtained during the initial site reconnaissance and literature review, prior experience with similar subsurface conditions encountered in the project vicinity and potential risk to structure performance. Detailed information about how to develop a laboratory testing program and the type of tests required is presented in previous sited reference or refer to a soils textbook for a more detailed description of soil tests.

Laboratory tests of undisturbed samples provide a more accurate assessment of soil settlement and structural properties. Unconfined compression tests and other tests are employed to measure the undrained shear strength and to estimate pile shaft resistance in clay soils by assuming:

$$c = \frac{q_u}{2}$$

Where:

c	=	cohesion of soil
q_u	=	unconfined compression strength

It is worthy to note that pile shaft resistance is a function of multiple parameters, including but not limited to stress state, depth, soil type and foundation type.

In addition to the tests of subsurface materials, a geological and/or geophysical study may be conducted to give such geological aspects as petrology, rock structure, rock quality, stratigraphy, vegetation and erosion. This can include in-situ and laboratory testing of selected samples, as well as utilizing non-destructive geophysical techniques, such as seismic refraction, electromagnetic or ground penetrating radar (GPR)

Boring and testing data analysis, along with consideration of the geology and terrain, allow the geotechnical engineer to present the following in the bridge SIR:

- The preferred type of substructure foundation (i.e. shallow or deep).
- The factored bearing resistance for shallow foundations.
- The settlement for the shallow foundations.

- If piles are required, recommend the most suitable type and the support values (shaft resistance and point resistance) furnished by the different soil strata.

- A discussion of any geotechnical issues that may affect construction.

- The presence and effect of water, including discussion of dewatering impact and cut-slope impact under abutments.

When piles are recommended, suitable pile types, estimated length requirements, pile drivability and design loads are discussed. Adverse conditions existing at abutments due to approach fills being founded on compressible material are pointed out, and recommended solutions are proposed. Unfactored resistance values at various elevations are given for footing foundation supports. Problems associated with scour, tremie seals, cofferdams, settlement of structure or approach fill slopes and other conditions unique to a specific site are discussed as applicable.

10.3 Soil Classification

The total weight of the structure plus all of the forces imposed upon the structure is carried by the foundation soils. There are many ways to classify these soils for foundation purposes. An overall geological classification follows:

1. Bedrock - This is igneous rock such as granite; sedimentary rock such as limestone, sandstone and shale; and metamorphic rock such as quartzite or marble.

2. Glacial soils (Intermediate Geo Material- IGM) - This wide variety of soils includes granular outwash, hard tills, bouldery areas and almost any combination of soil that glaciers can create and are typically defined to have a SPT number greater than 50.

3. Alluvial soils - These are found in flood plains and deltas along creeks and rivers. In Wisconsin, these soils normally contain large amounts of sand and silt. They are highly stratified and generally loose. Pockets of clay are found in backwater areas.

4. Residual soils - These soils are formed as a product of weathering and invariably reflect the parent bedrock material. They may be sands, silts or clay.

5. Lacustrine soils - These soils are formed as sediment and are deposited in water environments. In Wisconsin, they tend to be clayey. One example of these soils is the red clay sediments around Lakes Superior and Michigan.

6. Gravel, cobbles and boulders - These are particles that have been dislodged from bedrock, then transported and rounded by abrasion. Some boulders may result from irregular weathering.

Regardless of how the materials are formed, for engineering purposes, they are generally broken into the categories of bedrock, gravel, sand, silt, clay or a combination of these. The behavioral characteristics of any soil are generally based on the properties of the major constituent(s). Listed below are some properties associated with each of these material types.

1. Sand - The behavior of sand depends on grain size, gradation, density and water conditions. Sand scours easily, so foundations on sand must be protected in areas subject to scour.

2. Silt - This is a relatively poor foundation material. It scours and erodes easily and causes large volume changes when subject to frost.

3. Clay - This material needs to be investigated very carefully for use as a bearing material. Long-term consolidation may be an issue.

4. Bedrock - This is generally the best foundation material. Wisconsin has shallow weathered rock in many areas of the state. Weathered granite and limestone become sands. Shale and sandstone tend to weather more on exposure.

5. Mixture of soils - This is the most common case. The soil type with predominant behavior has the controlling name. For example, a soil composed of sand and clay is called sandy clay if the clayey fraction controls behavior.

10.4 Site Investigation Report

The following is a sample of a Site Investigation Report for a two-span bridge and retaining wall. The subsurface exploration drawing is also submitted with the reports.

CORRESPONDENCE/MEMORANDUM ___________________State of Wisconsin

DATE: February 17, 2015

TO: Casey Wierzchowski, P.E.
Southeast Region Soils Engineer

FROM: Jeffrey D Horsfall, P.E.
Geotechnical Engineer

SUBJECT: **Site Investigation Report**
Project I.D. 1060-33-16
B-40-0880
Center Street over USH 45
Milwaukee County

Attached is the Site Investigation Report for the above project.

Please call if you have any questions.

Attachments

cc: Southeast Region (via e-mail)
Bureau of Structures, Structures Design (via e-submit)
Geotechnical File (original)

Site Investigation Report
Project I.D. 1060-33-16
Structure B-40-0880
Center Street over USH 45
Milwaukee County
February 17, 2015

1. GENERAL

The project is Center Street over USH 45, Milwaukee County. The proposed structure has two spans and will replace the existing structure with four spans (B-40-284). The existing structure is supported on spread footings with an allowable bearing capacity of 5,000 psf. The end slope in front of the abutments is to be supported with MSE walls with precast concrete panels. The current topography near the proposed structure is a rolling terrain in an urban area.

The Southeast Region requested that the Geotechnical Engineering Unit evaluate the foundation support for the proposed new structure. The following report presents results of the subsurface investigation, design evaluation, findings, conclusions, and recommendations.

2. SUBSURFACE CONDITIONS

Wisconsin Department of Transportation contracted with Gestra to completed one boring and PSI, Inc. to complete three borings near the proposed structure. Samples were collected in the borings with a method conforming to AASHTO T-206, Standard Penetration Test, in October and November 2014, using automatic hammers (with an efficiency ranging from 84 percent (Gestra) to 69 percent (PSI)). Attachment 1 presents tables showing the summary of subsurface conditions logged in the borings at this site and at the time of drilling for the structure. Attachment 2 presents a figure that illustrates the boring locations and graphical representations of the boring logs. The original borings logs are available at the Geotechnical Engineering Unit and will be made available upon request.

The following describes subsurface conditions in the four borings:

> 0.7 feet of topsoil or 1.0 feet to 2.0 feet of pavement structure, overlying
> 0.0 feet to 7.0 feet of brown, dense to very dense, fine to course, sand and gravel, overlying
> 20.0 feet to 43.0 feet of brown to gray, medium hard, clay, some silt, trace sand, overlying
> 0.0 feet to 8.0 feet of gray, loose to dense, fine sand, little silt, overlying
> 0.0 feet to 26.0 feet of gray, medium hard, clay, some silt, trace sand, overlying
> Gray, very hard, clay and silt, some gravel

The observed groundwater elevation at the time of drilling ranged from 714 feet to 732 feet as determined by the drillers describing the samples as wet. However, not all of the borings encountered samples that were wet.

3. ANALYSIS ASSUMPTIONS

Foundation analyses are separated into shallow foundations (spread footings) and deep foundations (piling supports). The analyses used the following assumptions:

Shallow Foundation

1. The groundwater elevation ranged from 714 feet to 732 feet.
2. The base of the foundations are at the following elevations

Table 1: Foundation Elevations	
West Abutment	755.9 feet
Pier	733.3 feet
East Abutment	754.4 feet

3. The abutment end slopes are MSE Walls with precast panel facing.
4. The width of the pier footing is 10 feet and the width of the abutment footing is 6 feet.
5. The resistance factor of 0.55 for the factored bearing resistance.

Pile Supported Deep Foundation

1. Soil pressures for displacement piles are based upon a 10 3/4-inch diameter cast-in-place pile.
2. The groundwater elevation ranged from 714 feet to 732 feet.
3. Table 1 presents elevations at the base of the foundations.
4. Nominal soil pressures determined using the computer program APILE.
5. The drivability evaluation was performed using the computer program GRLWEAP.

The design shear strength, cohesion and unit weight for this analyses are presented latter in this report. The values are based upon empirical formulas for internal friction angles using blow counts from the AASHTO T-206 Standard Penetration Test results and the effective overburden pressure for the granular soils, the pocket penetrometer values for the cohesive soils and published values for the bedrock.

4. RESULTS OF ANALYSIS

Shallow Foundation

The results of the shallow foundation evaluation indicated that the factored bearing resistance was 6,000 psf for the west abutment and east abutment and 5,000 psf for the pier. The soils are relatively uniform. The estimated settlement from the bridge loads at the abutments and piers was excessive. The time for settlement would occur over a relatively long period of time.

Deep Foundation

Table 2 shows estimated nominal skin friction and end bearing values for deep foundation pilings.

Drivability

The drivability evaluation used a Delmag D 16-32 diesel hammer to determine if the pile would be overstressed during pile installation. The results of the evaluation indicated that 10 x 42 H-pile at the abutments and the 12 x 53 H-piles at the pier should not be overstressed.

Lateral Earth Pressure

The lateral earth pressure for the backfill material will exert 40 psf for sandy soils. The backfill material will be granular, free draining and locally available.

Table 2: Soil Parameters and Foundation Capacities

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Skin Friction[1] (psf)	End Bearing[1] (psf)
B-40-0880 West Abutment (B-1)					
MSE Wall (Elevation 755.9 ft – 738.6 ft)	30	0	120	NA	NA
Clay, gray, trace gravel (Elevation 738.6 ft – 733.4 ft)	0	3,000	125	640	19,100
Clay, gray, trace gravel (Elevation 733.4 ft – 729.4 ft)	0	2,500	120	1,075	21,700
Clay, gray, trace gravel (Elevation 729.4 ft – 717.4 ft)	0	2,000	120	1,370	17,900
Clay and Silt, gray, trace sand and gravel (Elevation 717.4 ft – 705.4 ft)	0	4,500	135	1,210	40,500
Silt, gray, trace sand (Elevation 705.4 ft – 700.4 ft)	0	2,000	120	1,720	17,900
Silt, gray, some sand, trace gravel (Elevation 700.4 ft and below)	0	25,000	135	NA	Refusal
B-40-0880 Pier (B-1Gestra)					
Clay, brown to gray, trace sand, trace gravel (Elevation 733.3 ft – 731.7 ft)	0	2,000	120	340	15,800
Clay, gray, trace gravel (Elevation 731.7 ft – 715.7 ft)	0	3,000	125	930	27,000
Silt, gray, trace gravel (Elevation 715.7 ft – 698.7 ft)	0	3,500	130	495	31,600
Silt, gray, trace gravel (Elevation 698.7 ft – 694.2 ft)	40	0	135	470	417,800
Silt, Sand, Gravel, gray (Elevation 694.2 ft and below)	0	25,000	135	NA	Refusal

1. Skin friction and end bearings vales are the nominal capacities
2. NA - not applicable

Table 2: Soil Parameters and Foundation Capacities					
Soil Description	**Friction Angle (degrees)**	**Cohesion (psf)**	**Unit Weight (pcf)**	**Skin Friction[1] (psf)**	**End Bearing[1] (psf)**
B-40-0880 East Abutment (B-2 and B-3)					
MSE Wall (Elevation 754.4 ft – 741.5 ft)	30	0	120	NA	NA
Clay, gray, trace gravel (Elevation 741.5 ft – 732.5 ft)	0	2,500	125	920	22,500
Sand, gray, some silt (Elevation 732.5 ft – 730.5 ft)	36	0	130	620	45,900
Sand, gray, some silt (Elevation 730.5 ft – 728.5 ft)	30	0	115	340	19,700
Clay, gray, trace sand, trace gravel (Elevation 728.5 ft – 717.5 ft)	0	2,500	125	2,380	22,500
Clay, gray, trace sand, trace gravel (Elevation 717.5 ft – 711.0 ft)	0	2,000	120	1,830	17,900
Silt, gray, trace sand (Elevation 711.0 ft – 702.5 ft)	33	0	125	890	50,000
Clay, gray (Elevation 702.5 ft – 692.5 ft)	0	3,000	125	1,730	27,000
Clay and Gravel, gray, some silt (Elevation 692.5 ft and below)	0	25,000	135	NA	Refusal

1. Skin friction and end bearings vales are the nominal capacities
2. NA - not applicable

5. FINDING AND CONCLUSIONS

The following findings and conclusions are based upon the subsurface conditions and analysis:

1. The following describes the subsurface conditions in the four borings:

   0.7 feet of topsoil or 1.0 feet to 2.0 feet of pavement structure, overlying
   0.0 feet to 7.0 feet of brown, dense to very dense, fine to course, sand and gravel, overlying
   20.0 feet to 43.0 feet of brown to gray, medium hard, clay, some silt, trace sand, overlying
   0.0 feet to 8.0 feet of gray, loose to dense, fine sand, little silt, overlying
   0.0 feet to 26.0 feet of gray, medium hard, clay, some silt, trace sand, overlying
   Gray, very hard, clay and silt, some gravel

2. The observed groundwater elevation at the time of drilling ranged from 714 feet to 732 feet as determined by the drillers describing the samples as wet.

3. The results of the shallow foundation evaluation indicated that the factored bearing resistance was 6,000 psf for the west abutment and east abutment and 5,000 psf for the pier. The soils are relatively uniform. The calculations used a resistance factor of 0.55.

4. The estimated settlement from the bridge loads on the shallow foundations would be excessive. The time for settlement would occur over a long period of time.

5. If used the support of the piles will occur in the very hard clay and silt. The pile tip elevation will range from 692 feet to 700 feet. The driven pile lengths will depend upon the type of pile hammer used and actual subsurface conditions encountered.

6. RECOMMENDATIONS

The following recommendations are based upon the findings and conclusions:

1. The recommended support system for the abutments are 10 x 42 H-piles driven to a “Required Driving Resistance” of 180 tons and for the pier footings are 12 x 53 H-piles driven to a “Required Driving Resistance” of 220 tons. Table 3 presents the estimated pile tip elevation for the piles. The actual driven length may be shorter due to the very hard clay.

   Table 3: Estimated H-Pile Tip Elevations

Substructure	Pile Type	Pile Tip Elevation
West Abutment	10 x 42 H-pile	700 feet
Pier	12 x 53 H-pile	694 feet
East Abutment	10 x 42 H-pile	692 feet

2. The field pile capacity should be determined by using the modified Gates dynamic formula. This method will use of a resistance factor of 0.50.

3. Pile points should be used to reduce the potential for damage during driving through the very hard clay and silts.

4. Shallow foundation should not be used based upon the anticipated settlement at the pier and the MSE walls at the abutments.

5. Granular 1 backfill should be used behind the abutments.

Attachment 1
Tables of Subsurface Conditions

B-40-0880 Subsurface Conditions							
B-1 Station 19+00.0 22.4 feet left of CE RL				B-1Gestra Station 20+11.3 38.2 feet left of CE RL			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count[1]**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
762.6	Pavement Structure			742.7	Pavement Structure	6	14
761.6	Clay, dark brown, trace sand and gravel (fill)	4	7	740.7	Clay, brown to gray, trace sand, trace gravel Qp=1.0 – 3.0	6,9, 9,13	12,17, 16,21
754.1	Clay, brown, some silt, trace sand and gravel Qp=3.0	18	25	731.7	Clay, gray, trace gravel Qp=3.0 – 4.0	9,10, 11,13, 14,12	14,15, 16,18, 19,15
749.6	Clay, gray, trace gravel Qp=1.75 – 3.5	15,13, 14	18,14, 15	715.7	Silt, gray, trace sand Qp=4.0	24,33, 31	27,36, 31
739.6	Clay, gray, trace gravel Qp=3.0 – 3.75	20,14, 18	21,14, 17	698.7	Silt, gray, with gravel Qp=4.5	50/6”	51/6”
733.6	Clay, gray, trace gravel Qp=2.0 – 2.5	23,29	22,26	694.2	Silt, Sand, Gravel, gray Qp=4.5	79, 50/2”	78, 48/2”
729.6	Clay, gray, trace gravel Qp=1.5 – 3.0	13,15, 24,17	12,13, 20,13	689.7	EOB		
717.6	Clay and Silt, gray, trace sand and gravel Qp=3.0 - 4.5+	66,67	49,47				
705.6	Silt, gray, trace sand Qp=1.5	28	18				
700.6	Silt, gray, some sand, trace gravel Qp=4.5+	78,42, 59, 60/4”	49,25, 34, 33/4”				
682.6	EOB						

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring.

B-40-0880 Subsurface Conditions							
B-3 Station 21+10.0 40.6 feet right of CE RL				**B-2 Station 21+14.8 23.3 feet left of CE RL**			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
759.4	Topsoil			760.5	Pavement Structure		
758.7	Sand, light brown to brown, fine to course, trace silt and gravel	14,13	32,24	759.5	Sand and Gravel, brown	31	49
755.4	Clay, brown, some silt, trace sand and gravel Qp=4.5 – 4.5+	14,32, 16,50	23,48, 22,65	752.5	Clay and Silt, brown, trace gravel Qp=2.5 – 3.0	11,15	15,18
747.4	Clay, gray, trace sand and gravel Qp=2.5 – 3.25	32,13, 14,15	40,15, 15,15	742.5	Clay, gray, trace gravel Qp=1.75 – 4.5+	18,22, 24,15, 19	19,23, 24,15, 18
730.4	Sand, gray, fine, little silt	29	27	732.5	Sand, gray, some silt	38	35
726.4	Sand, gray, fine, little silt	9	8	730.5	Sand, gray, some silt	9	8
722.4	Silt, gray, little sand, trace clay Qp=3.0	15	13	728.5	Clay, gray, trace sand and gravel Qp=2.5 – 3.0	22,14, 17,20, 21	20,12, 15,17, 17
719.4	EOB			711.0	Silt, gray, trace sand Qp=1.0	38	30
				702.5	Clay, gray Qp=1.75 – 3.0	21,27	16,20
				692.5	Clay and Gravel, gray, some silt Qp=4.5+	117, 108, 60/2’	85, 76, 41/2”
				680.5	EOB		

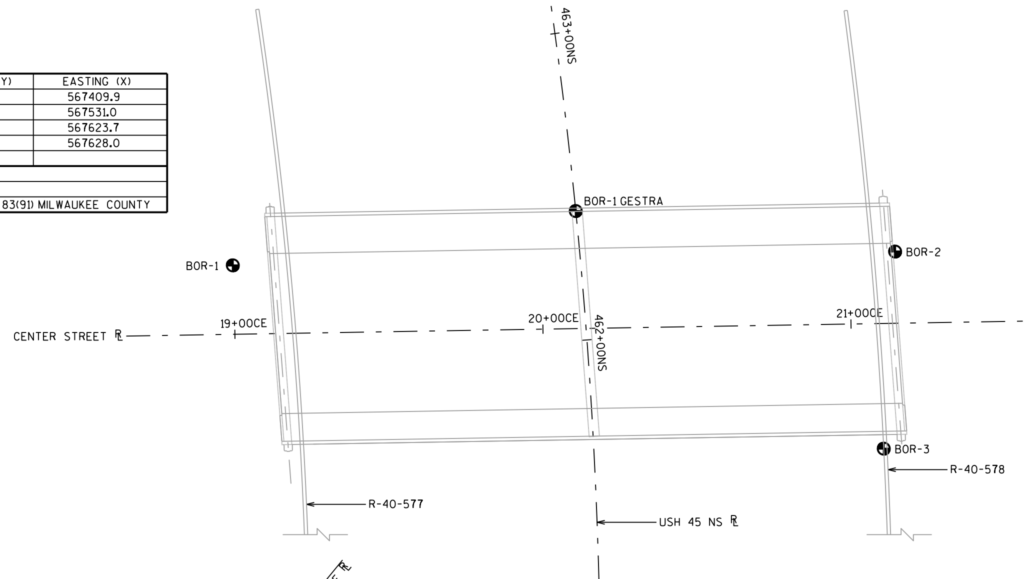
1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring.

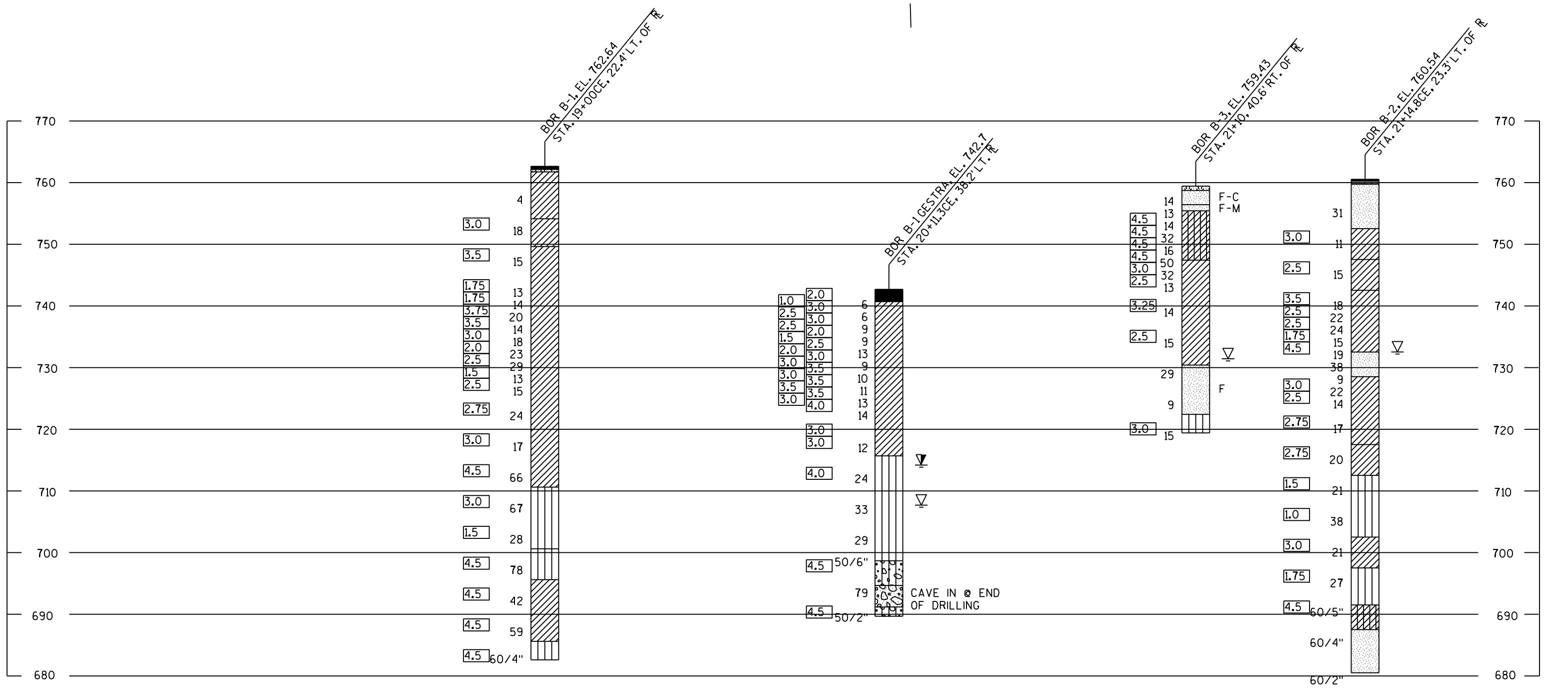
Attachment 2
Bridge Figure

ZOO INTERCHANGE, NORTH LEG
CENTER STREET OVER USH 45

BORING #	DATE COMPLETED	NORTHING (Y)	EASTING (X)
1	11/3/2014	310125.9	567409.9
GESTRA 1	10/16/2014	310131.3	567531.0
2	11/4/2014	310125.5	567623.7
3	11/5/2014	310040.4	567628.0

BORINGS COMPLETED BY: PSI/GESTRA
REPORT COMPLETED BY: WISDOT
ALL COORDINATES REFERENCED TO WCCS NAD 83(91) MILWAUKEE COUNTY





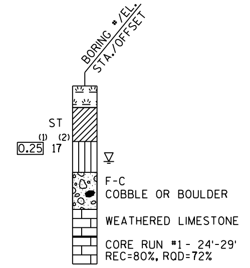
STATE PROJECT NUMBER

1060-33-16

MATERIAL SYMBOLS

ASPHALT | TOPSOIL | PEAT
CONCRETE | FILL | GRAVEL
SAND | CLAY | SILT
BOULDERS OR COBBLES | LIMESTONE | BEDROCK (UNKNOWN)
SHALE | SANDSTONE | IGNEOUS/META

LEGEND OF BORING



(1) UNCONFINED STRENGTH, AS DETERMINED BY A POCKET PENETROMETER (TSF)

(2) UNLESS OTHERWISE, SPECIFIED THE SPT 'N' VALUE IS BASED ON AASHTO T-206, STANDARD PENETRATION TEST. THE SPT 'N' VALUE PRESENTED HAS NOT BEEN CORRECTED FOR OVERBURDEN PRESSURE OR HAMMER EFFICIENCY.

GROUND WATER ELEVATION

AT TIME OF DRILLING
END OF DRILLING
AFTER DRILLING

ABBREVIATIONS

F-FINE M-MEDIUM C-COARSE ST-SHELBY TUBE

SUBSURFACE EXPLORATION FOR FOUNDATION DESIGN AND BIDDERS INFORMATION

BORINGS WERE COMPLETED AT POINTS APPROXIMATELY AS INDICATED ON THIS DRAWING TO OBTAIN INFORMATION CONCERNING THE CHARACTER OF SUBSURFACE MATERIALS FOUND AT THE SITE. BECAUSE THE INVESTIGATED DEPTHS ARE LIMITED AND THE AREA OF THE BORINGS IS VERY SMALL IN RELATION TO THE ENTIRE SITE, THE WISCONSIN DEPARTMENT OF TRANSPORTATION DOES NOT WARRANT SIMILAR SUBSURFACE CONDITIONS BELOW, BETWEEN, OR BEYOND THESE BORINGS. VARIATIONS IN SOIL CONDITIONS SHOULD BE EXPECTED AND FLUCTUATIONS IN GROUNDWATER LEVELS MAY OCCUR.

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-40-880

DRAWN BY PR | PLANS CK'D.

SUBSURFACE EXPLORATION

SHEET

SCALE =

8

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16	BORING ID: B-1
	WISDOT STRUCTURE ID: B-40-880-2	PAGE NO: 1 of 4

WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE: N43° 04.048'	LONGITUDE: W88° 03.229'
ROADWAY NAME: Center Street Over USH 45	DRILLING CONTRACTOR: PSI	DRILLING CONTRACTOR PROJECT NO: 0052853-7	NORTHING:	EASTING:
DATE STARTED: 11/03/14	CREW CHIEF: P. Rotaru	DRILL RIG: Freightliner	COORDINATE SYSTEM: Lat/Long	
DATE COMPLETED: 11/03/14	LOGGED BY: D. Zuydhoek	HOLE SIZE: 10 in	HORIZONTAL DATUM: WGS 1984	VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA	

STATION	OFFSET	TOWNSHIP:	RANGE:	SECTION:	1/4 SECTION:	1/4 1/4 SECTION:	SURFACE ELEVATION:
462+35	112.5' LT						762.64 ft

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
				0.5		ASPHALT, (5.5" Thick) 762.1						HSA	
				0.9		BASE COURSE, (5" Thick) 761.7	GPS						
				1–3		CLAY, Fill, Dark Brown, Soft, Trace Sand and Gravel	CL						
SPT 1	4	M	3-2-2-3 (4)	4									
				5–8									
				8.5		754.1							
SPT 2	24	M	5-6-12-17 (18)	9		CLAY, Brown, Very Stiff, Trace Sand and Gravel	CL	3.0					
				10–12									
				13.0		749.6							
SPT 3	24	M	8-8-7-11 (15)	14		CLAY, Gray, Very Stiff, Trace to Few Sand and Gravel	CL	3.5					
				15–18									
SPT 4	24	M	4-5-8-7 (13)	19				1.75					
SPT 5	24	M	4-6-8-8 (14)	21		Stiff		1.75					
SPT 6	24	M	6-9-11-10 (20)	23		Very Stiff		3.75					
SPT 7	24	M	6-6-8-11 (14)	25				3.5					
SPT 8	24	M	7-8-10-11 (18)	27				3.0					
SPT 9	24	M	11-11-12-12 (23)	29				2.0					

WATER LEVEL & CAVE-IN OBSERVATION DATA

▽ WATER ENCOUNTERED DURING DRILLING:	NE	CAVE - IN DEPTH AT COMPLETION:	NMR	WET ☐ DRY ☐
▼ WATER LEVEL AT COMPLETION:	NMR	CAVE - IN DEPTH AFTER 0 HOURS:	NMR	WET ☐ DRY ☐

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
SPT 10	24	M	16-15-14-17 (29)	31		CLAY, Gray, Very Stiff, Trace to Few Sand and Gravel		2.5					
SPT 11	24	M	5-6-7-10 (13)	32, 33		Stiff		1.5					
SPT 12	24	M	4-7-8-11 (15)	34, 35		Very Stiff		2.5					
				36, 37									
SPT 13	24	M	10-12-12-15 (24)	38, 39				2.75					
				40, 41, 42			CL						
SPT 14	24	M	6-7-10-13 (17)	43, 44				3.0					
				45, 46, 47								MR	
SPT 15	24	M	17-33-33-51 (66)	48, 49				4.5					
				50, 51, 52		Hard 52.0 710.6							
SPT 16	24	M	13-25-42-60 (67)	53, 54		SILT, Gray, Very Stiff, Trace Sand		3.0					
				55, 56, 57			ML						
SPT 17	24	M	8-12-16-18 (28)	58, 59				1.5					
				60, 61, 62		Stiff 62.0 700.6							
SPT 18	15	M	30-43-35-46 (78)	63, 64		SILT, Gray, Hard, Some Sand, Trace Gravel	ML	4.5					
				65, 66, 67		67.0 695.6							
SPT			11-20-22-	68		CLAY, Gray, Hard, Little Sand, Trace Gravel	CL						

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID:	1060-33-16	BORING ID:	B-1
	WISDOT STRUCTURE ID:	B-40-880-2	PAGE NO:	3 of 4

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
19	24	M	27 (42)	70		CLAY, Gray, Hard, Little Sand, Trace Gravel		4.5					
				71									
				72									
				73			CL						
SPT 20	24	M	15-23-36-31 (59)	74				4.5					
				75									
				76									
				77		77.0 — 685.6							
						SILT, Gray, Hard, Some Sand, Trace Gravel							
				78									
SPT 21	8	M	58-60/4"	79			ML	4.5					
				80		80.0 — 682.6							

End of Boring at 80.0 ft.

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16 WISDOT STRUCTURE ID: B-40-880	BORING ID: B-1 Gestra PAGE NO: 1 of 2

WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE:	LONGITUDE:
ROADWAY NAME: Center Street Over USH 45	DRILLING CONTRACTOR: GESTRA	DRILLING CONTRACTOR PROJECT NO:	NORTHING:	EASTING:
DATE STARTED: 10/16/14	CREW CHIEF: A. Woerpel	DRILL RIG: CME-75	COORDINATE SYSTEM: WCCS	
DATE COMPLETED: 10/16/14	LOGGED BY: A. Woerpel	HOLE SIZE: 3.25 in	HORIZONTAL DATUM: WCCS Milwaukee	VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA	
STATION 462+42 / OFFSET ON R/L	TOWNSHIP: / RANGE: / SECTION:	1/4 SECTION: / 1/4 1/4 SECTION:	SURFACE ELEVATION: 742.7 ft	

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
SPT 1	5	M	3-3 (6)	1		Asphalt Concrete 2.0 — 740.7		2.0				HSA	3 1/4 Hollowstem Auger
SPT 2	10	M	2-3-3-4 (6)	2–3		Moist Brown Clay with Trace Gravel Trace Sand		1.0 3.0					
SPT 3	22	M	2-3-6-7 (9)	4–5		Color Change To Gray Moist Clay Trace Gravel		2.5 3.0					
SPT 4	24	M	3-4-5-6 (9)	6–7				2.5 2.0					
SPT 5	24	M	3-6-7-9 (13)	8–9				1.5 2.5					
SPT 6	24	M	2-3-6-7 (9)	10–11				2.0 3.0					
SPT 7	24	M	2-4-6-7 (10)	12–13				3.0 3.5					
SPT 8	24	M	2-5-6-8 (11)	14–15		Moist Gray Clay Trace Gravel		3.0 3.5					
SPT 9	24	M	2-5-8-10 (13)	16–17				3.5 3.5					
SPT 10	24	M	2-5-9-10 (14)	18–19				3.0 4.0					
				20–23		Wet Pockets							
SPT 11	18	M	3-5-7 (12)	24				3.0 3.0					
				25–27		27.0 — 715.7							
				27–28		Moist Gray Silt With Trace Sand							
SPT 12	18	M	5-10-14 (24)	29				4.0					

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING: NMR	CAVE - IN DEPTH AT COMPLETION: NMR (WET / DRY)
WATER LEVEL AT COMPLETION: NMR	CAVE - IN DEPTH AFTER 0 HOURS: NMR (WET / DRY)

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

P:\GINT\WISDOT GINT PROJECTS\GINT_40\B-40-880.GPJ Center Street over US Highway 45 2/11/15

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
						Moist Gray Silt With Trace Sand							
				31									
				32									
				33									
SPT 13	18	M	10-14-19 (33)	34									
				35		Wet Silt And Sand Mix							
				36									
				37		Wet Gray Silt							
				38									
SPT 14	18	W	12-13-16 (29)	39									
				40									
				41									
				42									
				43		44.0 698.7							
SPT 15	12	M	20-50	44		Moist Silt With Gravel		4.5					
				45									
				46									
				47		48.0 694.7							
				48		Saturated Gray Sand & Gravel							
SPT 16	12	W	16-35-44 (79)	49									
				50									
				51		51.5 691.2							
				52		Moist Silt With Gravel							
						53.0 689.7							
SPT 17	2	M	50/2"	53		End of Boring at 53.0 ft.		4.5					

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16		BORING ID: B-2
	WISDOT STRUCTURE ID: B-40-880-3		PAGE NO: 1 of 4

WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE: N43° 04.048'	LONGITUDE: W88.03.181'
ROADWAY NAME: Center Street	DRILLING CONTRACTOR: PSI	DRILLING CONTRACTOR PROJECT NO: 0052853-4	NORTHING:	EASTING:
DATE STARTED: 11/04/14	CREW CHIEF: P. Rotaru	DRILL RIG: Freightliner	COORDINATE SYSTEM: Lat/Long	
DATE COMPLETED: 11/04/14	LOGGED BY: D. Zuydhoek	HOLE SIZE: 10 in	HORIZONTAL DATUM: WGS 1984	VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA	
STATION 462+20 / OFFSET 102' RT	TOWNSHIP: / RANGE: / SECTION:	1/4 SECTION: / 1/4 1/4 SECTION:	SURFACE ELEVATION: 760.54 ft	

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
						0.3 ASPHALT, (4" Thick) 760.2						HSA	
				1		0.6 CONCRETE, (3" Thick) 759.9	GPS						
						0.8 BASE COURSE, (3" Thick) 759.7							
				2		SAND, Brown, Dense, Some Gravel							
SPT 1	12	M	17-15-16-10 (31)	3–5			SP						
				6–8		8.0 752.5							
SPT 2	24	M	9-5-6-8 (11)	9		CLAY, Brown, Very Stiff, Trace Sand and Gravel	CL	3.0					
				10–13		13.0 747.5							
SPT 3	24	M	5-7-8-11 (15)	14		CLAY, Brown, Very Stiff, Trace Silt, Sand and Gravel	CL	2.5					
				15–18		18.0 742.5							
SPT 4	24	M	6-7-11-13 (18)	19		CLAY, Gray, Very Stiff, Trace Sand and Gravel		3.5					
SPT 5	24	M	12-10-12-12 (22)	21				2.5					
SPT 6	24	M	11-13-11-12 (24)	23			CL	2.5					
SPT 7	24	M	4-7-8-11 (15)	25		Stiff		1.75					
SPT 8	18	M	5-6-13-15 (19)	27		Hard 28.0 732.5		4.5					
SPT 9	24	W	19-22-16-16 (38)	29		SAND, Gray, Dense, Little Silt	SP						

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING: NMR	CAVE - IN DEPTH AT COMPLETION: NMR	WET ☐ DRY ☐
WATER LEVEL AT COMPLETION: NMR	CAVE - IN DEPTH AFTER 0 HOURS: NMR	WET ☐ DRY ☐

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
SPT 10	24	W	2-3-6-9 (9)	31		<u>SAND</u>, Gray, Dense, Little Silt Loose	SP						
				32		32.0 728.5							
SPT 11	24	W	6-9-13-15 (22)	33		<u>CLAY</u>, Gray, Very Stiff, Trace Sand and Gravel		3.0					
SPT 12	24	W	4-6-8-8 (14)	34, 35				2.5					
				36, 37, 38		Little Sand	CL						
SPT 13	24	W	5-6-11-12 (17)	39				2.75					
				40, 41, 42		43.0 717.5							
SPT 14	24	M	7-8-12-12 (20)	43, 44		<u>CLAY</u>, Gray, Very Stiff, Trace Gravel		2.75					
				45, 46, 47		48.0 712.5	CL						
SPT 15	24	W	6-9-12-19 (21)	48, 49		<u>SILT</u>, Gray, Stiff, Trace Sand		1.5				MR	
				50, 51, 52									
SPT 16	18	W	17-18-20-22 (38)	53, 54			ML	1.0					
				55, 56, 57		58.0 702.5							
SPT 17	24	W	5-8-13-16 (21)	58, 59		<u>CLAY</u>, Gray, Very Stiff, Trace Sand and Gravel		3.0					
				60, 61, 62		63.0 697.5	CL						
SPT 18	18	W	10-13-14-27 (27)	63, 64		<u>SILT</u>, Gray, Stiff, Trace Sand		1.75					
				65, 66, 67			ML						
SPT	17	W	37-57-	68		69.0 691.5		4.5					

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
19			60/5"			SILTY CLAY, Gray, Hard, Trace Sand and Gravel							
				70									
				71			CL-ML						
				72									
				73		73.0 687.5							
SPT 20	12	W	53-48-60/4"	74		SAND, Gray, Very Dense, Some Gravel, Trace Silt							
				75									
				76			SP						
				77									
SPT 21	2	W	60/2"	78									
				79									
				80		80.0 680.5							

End of Boring at 80.0 ft.

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16 WISDOT STRUCTURE ID: R-40-578-3		BORING ID: B-3 PAGE NO: 1 of 2	
WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE: N43° 04.034'	LONGITUDE: W88° 03.180'
ROADWAY NAME: Center Street Over USH 45	DRILLING CONTRACTOR: PSI	DRILLING CONTRACTOR PROJECT NO: 0052853-4	NORTHING:	EASTING:
DATE STARTED: 11/05/14	CREW CHIEF: M. Ball	DRILL RIG: Diedrich D-50	COORDINATE SYSTEM: Lat/Long	
DATE COMPLETED: 11/05/14	LOGGED BY: D. Zuydhoek	HOLE SIZE: 10 in	HORIZONTAL DATUM: WGS 1984	VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA	
STATION: 461+60 / OFFSET: 94' RT	TOWNSHIP: / RANGE:	SECTION: / 1/4 SECTION: / 1/4 1/4 SECTION:	SURFACE ELEVATION: 759.43 ft	

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
				0.7		TOPSOIL, (8" Thick) — 758.7							
SPT 1	18	D	3-5-9-10 (14)	1		SAND, Brown, Firm, Fine to Coarse, Trace Silt and Gravel	SP					HSA	
				3.0		756.4							
SPT 2	24	D	12-7-6-6 (13)	3		SAND, Light Brown, Firm, Fine to Medium	SP						
				4.0		755.4							
SPT 3	24	M	4-6-8-11 (14)	5		SILTY CLAY, Brown, Hard, Trace Sand and Gravel	CL-ML	4.5					
SPT 4	12	M	7-12-20-18 (32)	7				4.5					
SPT 5	24	M	5-6-10-12 (16)	9				4.5					
SPT 6	24	M	12-25-25-23 (50)	11				4.5					
				12.0		747.4							
SPT 7	24	M	18-15-17-17 (32)	13		CLAY, Gray, Very Stiff, Trace Sand and Gravel	CL	3.0					
SPT 8	24	M	4-6-7-7 (13)	15				2.5					
SPT 9	24	M	5-6-8-10 (14)	19				3.25					
SPT 10	24	M	9-7-8-8 (15)	24				2.5					
				29.0		730.4							
SPT 11	24	W	28-16-13-13 (29)	29		SAND, Gray, Firm, Fine, Little Silt	SP						

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING: NMR	CAVE - IN DEPTH AT COMPLETION: NMR (WET / DRY)
WATER LEVEL AT COMPLETION: NMR	CAVE - IN DEPTH AFTER 0 HOURS: NMR (WET / DRY)

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
				31		SAND, Gray, Firm, Fine, Little Silt	SP						
				32									
SPT 12	24	W	3-3-6-6 (9)	33–35									
				35		Loose							
				36									
				37		37.0 722.4							
				37		SILT, Gray, Very Stiff, Little Sand, Trace Clay	ML						
SPT 13	24	W	4-8-7-8 (15)	38–40		40.0 719.4		3.0					

End of Boring at 40.0 ft.

CORRESPONDENCE/MEMORANDUM ___________________State of Wisconsin

DATE: April 10, 2015

TO: Casey Wierzchowski, P.E.
Southeast Region Soils Engineer

FROM: Jeffrey D Horsfall, P.E.
Geotechnical Engineer

SUBJECT: **Site Investigation Report**
Project I.D. 1060-33-16
R-40-0577
Center Street over USH 45
(West Abutment B-40-0880)
Milwaukee County

Attached is the Site Investigation Report for the above project.

Please call if you have any questions.

Attachments

cc: Southeast Region (via e-mail)
Bureau of Structures, Structures Design (via e-submit)
Geotechnical File (original)

Site Investigation Report
Project I.D. 1060-33-16
Structure R-40-0577
Center Street over USH 45
(West Abutment B-40-0880)
Milwaukee County
April 10, 2015

1. GENERAL

The project is a retaining wall located along the west side of USH 45 near Center Street, Milwaukee County. A portion of the proposed retaining wall supports the West Abutment of B-40-0880. Table 1 presents the location of the wall compared to the wall stationing

Table 1: Wall Locations		
USH 45 Roadway Station	Wall Station	Description
457+75.0, 92.0’ left	10+00.0	Beginning of Wall and supports side slope
463+22.0, 94.0’ left	12+33.8	End of Wall and supports side slope

The maximum exposed height is 24.9 feet. The proposed wall type is a MSE wall with precast concrete panels. Aesthetics is a key item to consider in the evaluation of the wall. A portion of the wall is located within a cut section of the roadway. Topography in the general vicinity is urban with a bridge approach located near the wall.

The Southeast Region requested that the Geotechnical Unit evaluate a MSE wall with precast concrete panels. The following report presents the results of the subsurface investigation, the design evaluation, the findings, the conclusions and the recommendations.

2. SUBSURFACE CONDITIONS

Wisconsin Department of Transportation contracted with PSI to completed three borings near the proposed wall. Samples were collected with a method conforming to AASHTO T-206, Standard Penetration Test, using an automatic hammer. The purpose of the borings was to define subsurface soil conditions at this site. Soil textures in the boring logs were field identified by the drillers. Attachment 1 presents tables showing the summaries of subsurface conditions logged in the borings at this site and at the time of drilling for the retaining wall. Attachment 2 presents a figure that illustrates the boring locations and graphical representations of the boring logs. The original borings logs are available at the Central Office Geotechnical Engineering Unit and will be made available upon request.

The following describes the subsurface conditions in the three borings:

0.0 feet to 1.0 foot of pavement structure, overlying
0.0 feet to 7.5 feet of dark brown, soft, clay, trace sand and gravel (fill, B-1), overlying
3.0 feet to 36.5 feet of brown, medium hard to hard, clay, trace sand and gravel, overlying
5.0 feet to 25.0 feet of brown to gray, fine to medium, firm to very dense, sand or silt, trace gravel, overlying
Gray, very hard, silt and clay, little sand, trace gravel

Generally, groundwater was not encountered in the borings at the time of drilling.

3. ANALYSIS ASSUMPTIONS

Chapter 14 of the WisDOT Bridge Manual describe ten different types of retaining structures: reinforced cantilever, gabion, post and panel, sheet pile, modular block gravity, mechanically stabilized earth (MSE) with 4 types of facings, and modular bin and crib walls. Geotechnical Engineering Unit procedures require that the wall alternatives requested by the region be evaluated to determine the feasibility at a particular location, from a geotechnical standpoint.

Table 2 presents the design soil parameters utilized for the analyses, which approximate the conditions at B-7, B-6 and B-1.

Table 2: Soil Parameters

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)
Granular Backfill Within the wall in the reinforcing zone	30	0	120
Fill Behind and below the reinforcing zone	31	0	120
B-7, 11+00			
Silt, gray, trace sand and gravel (Elevation 745.9 ft – 741.4 ft)	0	4,500	135
Sand, gray, fine to medium (Elevation 741.4 ft – 737.4 ft)	36	0	135
Silt, gray, trace sand, trace clay (Elevation 737.4 ft – 723.4 ft)	0	2,500	125
Silt, gray, trace sand, trace clay (Elevation 723.4 ft – 716.4 ft)	0	4,500	135
B-6, 12+00			
Silt, gray, trace clay, trace sand, trace gravel (Elevation 743.4 ft – 738.4 ft)	0	4,500	135
Sand, gray, fine to medium (Elevation 738.4 ft – 732.4 ft)	32	0	120
Clay, gray, little silt, trace sand, trace gravel (Elevation 732.4 ft – 710.4 ft)	0	3,000	128
Clay, gray, little silt, trace sand, trace gravel (Elevation 710.4 ft – 709.4 ft)	0	4,500	135
B-1, 14+60			
Clay, gray, trace gravel (Elevation 738.6 ft – 733.6 ft)	0	3,000	125
Clay, gray, trace gravel (Elevation 733.6 ft – 729.6 ft)	0	2,500	120
Clay, gray, trace gravel (Elevation 729.6 ft – 717.6 ft)	0	2,000	120
Clay and Silt, gray, trace sand and gravel (Elevation 717.6 ft – 705.6 ft)	0	4,500	135

Table 2: Soil Parameters			
Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)
B-1, 14+60 (continued)			
Clay, gray, trace sand (Elevation 705.6 ft – 700.6 ft)	0	2,000	120
Silt, gray, some sand, trace gravel (Elevation 700.6 ft and below)	0	25,000	135

The typical wall section used in the analyses had an **exposed** height that varies from 8.7 feet to 24.9 feet. The following assumptions are also included in the analyses:

1. The slope in front and behind the wall is horizontal.
2. Groundwater was not used in the analyses.
3. The granular backfill is free draining and will not become saturated.
4. The minimum embedment depth is 1.5 feet.
5. A surcharge load of 240 psf is included to model pedestrian and lightweight construction equipment.
6. An additional surcharge load equivalent to the weight of the soil behind the abutment is also included in the design.
7. Global stability factor of safety was determined by the computer program STABLPRO.
8. Bearing resistance is determined by Terzaghi's bearing capacity equation.
9. Settlement of the foundation on cohesionless and cohesive soil is based upon methods described in the FHWA Soils and Foundations Manual.

4. RESULTS OF ANALYSIS

The Geotechnical Unit evaluated a MSE wall with precast concrete facing for the project. The wall was evaluated for sliding, overturning, bearing resistance, global stability and settlement.

Table 3 presents the results of the evaluation and the Capacity to Demand Ratio (CDR).
The exposed wall height examined varied from 8.7 feet to 24.9 feet. The length of reinforcement for the wall is determined by meeting the eccentricity requirements (B/4>e) and a minimum embedment length of 8 feet.

The results of the evaluation indicated that if the sliding and bearing resistance requirements are met, then the eccentricity is also met. The global stability of the wall at the critical location was stable with a CDR of greater than 1.0.

The settlement of the foundation was estimated to be less than 1 inches and should occur within years of loading of the wall. The subsurface soils are relatively uniform; therefore, differential settlement should not be an issue.

Table 3: Results of MSE Wall External Stability Evaluation				
Dimensions				
Wall Height (feet)[1]	10.2	13.2	18.8	26.4
Exposed Wall Height (feet)	8.7	11.7	17.3	24.9
Length of Reinforcement (feet)[3]	8.0	9.2	17.4	18.5
Length of Rein. / Wall Height	NA	0.70	0.93	0.70
Wall Station	11+00.0	12+00.0	14+50.0	14+67.2
Boring Used	B-7	B-6	B-1	B-1
Capacity to Demand Ratio (CDR)[4]				
Sliding (CDR > 1.0)	1.4	1.3	1.0	1.5
Eccentricity (CDR > 1.0)	1.5	1.2	1.0	1.3
Global Stability (CDR > 1.0)	NA	NA	2.1	NA
Bearing Resistance (CDR > 1.0)	2.4	1.8	1.1	1.1
Required Bearing Resistance (psf)	6,000	6,000	7,000	7,000

1. The wall height includes an embedment of 1.5 feet.
2. The wall stability evaluation included a surcharge load that was equal to the weight of the soil behind the abutment.
3. The length of reinforcement is the minimum required length.
4. CDR requirements and load and resistance factors are presented in Chapter 14 of the Bridge Manual.
5. NA not applicable, global slope stability was evaluated at the critical wall location.

5. *FINDINGS AND CONCLUSIONS*

The following findings and conclusions are based upon the subsurface conditions and the analysis:

1. The following describes the subsurface conditions in the three borings:

   0.0 feet to 1.0 foot of pavement structure, overlying
   0.0 feet to 7.5 feet of dark brown, soft, clay, trace sand and gravel (fill, B-1), overlying
   3.0 feet to 36.5 feet of brown, medium hard to hard, clay, trace sand and gravel, overlying
   5.0 feet to 25.0 feet of brown to gray, fine to medium, firm to very dense, sand or silt, trace gravel, overlying
   Gray, very hard, silt and clay, little sand, trace gravel

2. The groundwater was not encountered in the investigation.

3. Table 3 presents the results of the external stability evaluation and shows that if the sliding and bearing resistance requirements are satisfied, then the eccentricity and global stability will also be satisfied.

4. Settlement of the foundation was estimated to be less than 2 inches and should occur within months of loading of the wall. The subsurface soils are relatively uniform; therefore, differential settlement should not be an issue.

6. RECOMMENDATIONS

The following recommendations are based upon the findings and conclusions:

1. The MSE wall with precast concrete panels will achieve the external stability factors of safety if the sliding and bearing resistance requirements are met. Table 3 presents the minimum length of the reinforcement at the locations evaluated. In the area of the wall that supports the abutment, the ratio of length of reinforcement to total height of wall should be increased from 0.70 to 0.93.

2. The contractor should remove 6-inches of topsoil and silt and clay below the reinforcing zone and replace with granular fill in the areas that the topsoil and silt and clay are encountered.

3. The backfill behind the MSE wall with precast concrete facing should be granular and free draining.

4. The Southeast Region soils engineer should review the fill subsurface conditions prior to construction of the wall.

Attachment 1
Tables of Subsurface Conditions

Subsurface Conditions: R-40-0577							
B-7 Station 458+75 85.5 feet left of USH 45 RL				**B-6 Station 459+75 85.5 feet left of USH 45 RL**			
Estimated Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Estimated Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
751.4	Clay, brown, trace sand and gravel Qp=3.5	9	20	749.4	Clay, brown, trace sand and gravel Qp=2.25 – 2.5	7,12, 8	16,22, 13
748.4	Sand, brown, fine to medium, trace clay	18	33	743.9	Silt, gray, trace clay, trace sand, trace gravel Qp=4.5 – 4.5+	42,26	63,36
747.4	Silt, gray, trace sand and gravel Qp=3.0 – 4.5+	36,56, 62	58,82, 85	738.4	Sand, gray, fine to medium	12,31, 26	16,39, 31
741.4	Sand, gray, fine to medium	55,47	71,57	732.4	Clay, gray, little silt, trace sand, trace gravel Qp=3.25 – 4.5	23,17, 15,18	25,17, 14,16
737.4	Silt, gray, trace sand, trace clay Qp=2.5 – 4.5+	18,25, 18	21,27, 18	710.4	Clay, gray, little silt, trace sand, trace gravel Qp=3.5	43	35
723.4	Silt, gray, trace sand, trace clay Qp=3.5	108, 60/4”	100, 51/4”	709.4	EOB		
716.4	EOB						

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring.
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2.
4. EOB is the end of boring.

Subsurface Conditions: R-40-0577			
B-1 Station 462+35.0 112.5 feet left of USH 45 RL			
Estimated Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
762.6	Pavement Structure		
761.6	Clay, dark brown, trace sand and gravel (fill)	4	7
754.1	Clay, brown, some silt, trace sand and gravel Qp=3.0	18	25
749.6	Clay, gray, trace gravel Qp=1.75 – 3.5	15,13,14	18,14,15
739.6	Clay, gray, trace gravel Qp=3.0 – 3.75	20,14,18	21,14,17
733.6	Clay, gray, trace gravel Qp=2.0 – 2.5	23,29	22,26
729.6	Clay, gray, trace gravel Qp=1.5 – 3.0	13,15,24,17	12,13,20,13
717.6	Clay and Silt, gray, trace sand and gravel Qp=3.0 - 4.5+	66,67	49,47
705.6	Silt, gray, trace sand Qp=1.5	28	18
700.6	Silt, gray, some sand, trace gravel Qp=4.5+	78,42,59, 60/4”	49,25,34, 33/4”
682.6	EOB		

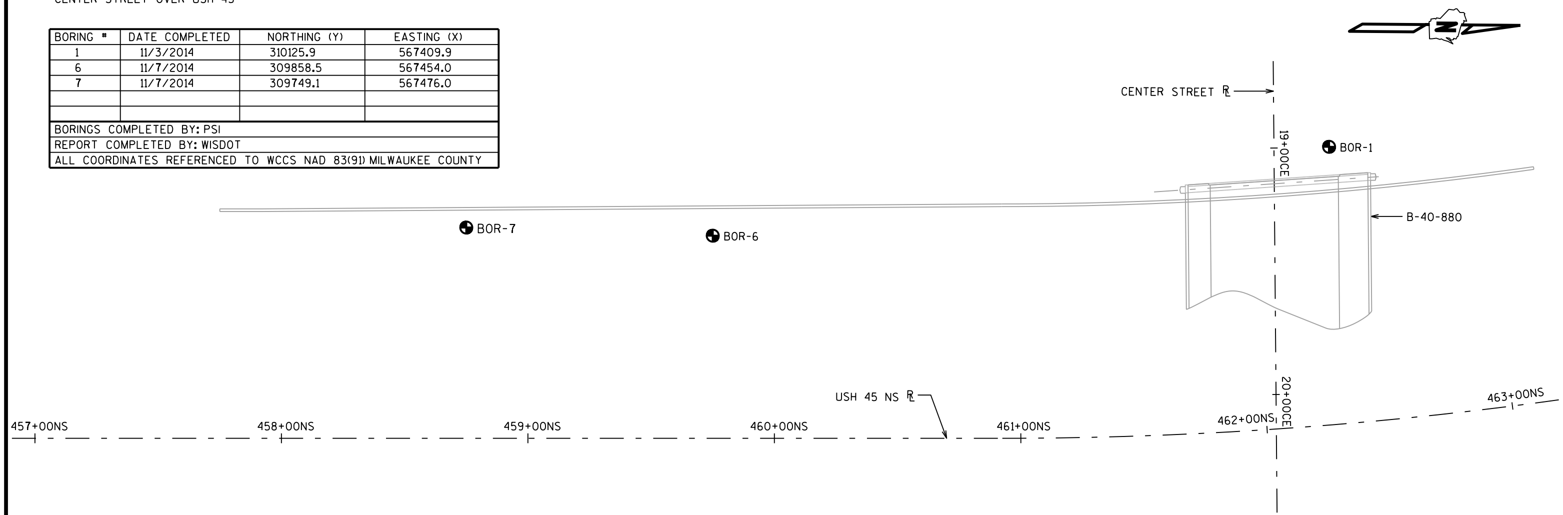
1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring.
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2.
4. EOB is the end of boring.

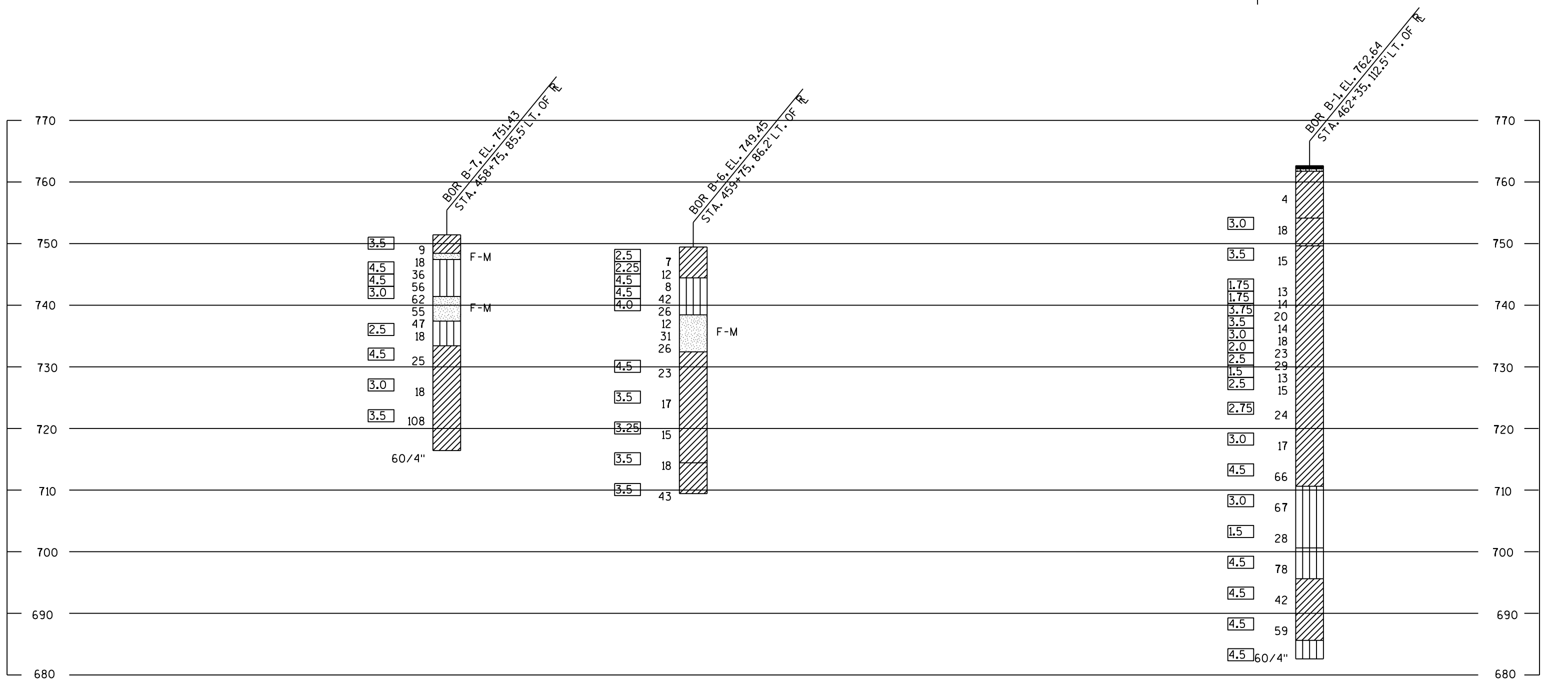
Attachment 2
Wall Figure

ZOO INTERCHANGE, NORTH LEG
CENTER STREET OVER USH 45

BORING #	DATE COMPLETED	NORTHING (Y)	EASTING (X)
1	11/3/2014	310125.9	567409.9
6	11/7/2014	309858.5	567454.0
7	11/7/2014	309749.1	567476.0

BORINGS COMPLETED BY: PSI
REPORT COMPLETED BY: WISDOT
ALL COORDINATES REFERENCED TO WCCS NAD 83(91) MILWAUKEE COUNTY





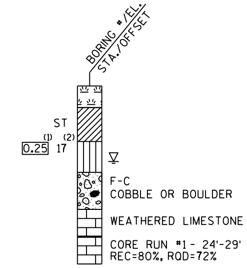
STATE PROJECT NUMBER

1060-33-16

MATERIAL SYMBOLS

ASPHALT, TOPSOIL, PEAT, CONCRETE, FILL, GRAVEL, SAND, CLAY, SILT, BOULDERS OR COBBLES, LIMESTONE, BEDROCK (UNKNOWN), SHALE, SANDSTONE, IGNEOUS/META

LEGEND OF BORING



(1) UNCONFINED STRENGTH, AS DETERMINED BY A POCKET PENETROMETER (TSF)

(2) UNLESS OTHERWISE SPECIFIED THE SPT 'N' VALUE IS BASED ON AASHTO T-206, STANDARD PENETRATION TEST. THE SPT 'N' VALUE PRESENTED HAS NOT BEEN CORRECTED FOR OVERBURDEN PRESSURE OR HAMMER EFFICIENCY.

GROUND WATER ELEVATION

- AT TIME OF DRILLING
- END OF DRILLING
- AFTER DRILLING

ABBREVIATIONS

F-FINE M-MEDIUM C-COARSE ST-SHELBY TUBE

SUBSURFACE EXPLORATION FOR FOUNDATION DESIGN AND BIDDERS INFORMATION

BORINGS WERE COMPLETED AT POINTS APPROXIMATELY AS INDICATED ON THIS DRAWING TO OBTAIN INFORMATION CONCERNING THE CHARACTER OF SUBSURFACE MATERIALS FOUND AT THE SITE. BECAUSE THE INVESTIGATED DEPTHS ARE LIMITED AND THE AREA OF THE BORINGS IS VERY SMALL IN RELATION TO THE ENTIRE SITE, THE WISCONSIN DEPARTMENT OF TRANSPORTATION DOES NOT WARRANT SIMILAR SUBSURFACE CONDITIONS BELOW, BETWEEN, OR BEYOND THESE BORINGS. VARIATIONS IN SOIL CONDITIONS SHOULD BE EXPECTED AND FLUCTUATIONS IN GROUNDWATER LEVELS MAY OCCUR.

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE R-40-577

DRAWN BY PR | PLANS CK'D.

SUBSURFACE EXPLORATION

SHEET

8

SCALE =

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Table of Contents

11.1 General

11.1.1 Overall Design Process

The overall foundation support design process requires an iterative collaboration to provide cost-effective constructible substructures. Input is required from multiple disciplines including, but not limited to, structural, geotechnical and design. For a typical bridge design, the following four steps are required (see 6.2):

1. Structure Survey Report (SSR) – This design step results in a very preliminary evaluation of the structure type and approximate location of substructure units, including a preliminary layout plan.

2. Site Investigation Report – Based on the Structure Survey Report, a Geotechnical Investigation (see Chapter 10 – Geotechnical Investigation) is required, including test borings to determine foundation requirements. A hydraulic analysis is also performed at this time, if required, to assess scour potential and maximum scour depth. The Site Investigation Report and Subsurface Exploration Drawing are used to identify known constraints that would affect the foundations in regard to type, location or size and includes foundation recommendations to support detailed structural design. Certain structure sites/types may require the preliminary structure plans (Step 3) prior to initiating the geotechnical site investigation. One example of this is a multi-span structure over water. See 6.2 for more information.

3. Preliminary Structure Plans – This design step involves preparation of a general plan, elevation, span arrangement, typical section and cost estimate for the new bridge structure. The Site Investigation Report is used to identify possible poor foundation conditions and may require modification of the structure geometry and span arrangement. This step may require additional geotechnical input, especially if substructure locations must be changed.

4. Final Contract Plans for Structures – This design step culminates in final plans, details, special provisions and cost estimates for construction. The Subsurface Exploration sheet(s) are part of the Final Contract Plans. Unless design changes are required at this step, additional geotechnical input is not typically required to prepare foundation details for the Final Contract Plans.

11.1.2 Foundation Type Selection

The following items need to be assessed to select site-specific foundation types:

- Magnitude and direction of loads.
- Depth to suitable bearing material.
- Potential for liquefaction, undermining or scour.
- Frost potential.

- Performance requirements, including deformation (settlement), lateral deflection, global stability and resistance to bearing, uplift, lateral, sliding and overturning forces.

- Ease, time and cost of construction.

- Environmental impact of design and construction.

- Site constraints, including restricted right-of-way, overhead and lateral clearance, construction access, utilities and vibration-sensitive structures.

Based on the items listed above, an assessment is made to determine if shallow or deep foundations are suitable to satisfy site-specific needs. A shallow foundation, as defined in this manual, is one in which the depth to the bottom of the footing is generally less than or equal to twice the smallest dimension of the footing. Shallow foundations generally consist of spread footings but may also include rafts that support multiple columns and typically are the least costly foundation alternative.

Shallow foundations are typically initially considered to determine if this type of foundation is technically and economically viable. Often foundation settlement and lateral loading constraints govern, rather than bearing capacity. Other significant considerations for selection of shallow foundations include requirements for cofferdams, bottom seals, dewatering, temporary excavation support/shoring, over-excavation of unsuitable material, slope stability, available time to dissipate consolidation settlement prior to final construction, scour susceptibility, environmental impacts and water quality impacts. Shallow foundations may not be economically viable when footing excavations exceed 10 to 15 feet below the final ground surface elevation.

When shallow foundations are not satisfactory, deep foundations are considered. Deep foundations can transfer foundation loads through shallow deposits to underlying deposits of more competent deeper bearing material. Deep foundations are generally considered to mitigate concerns about scour, lateral spreading, excessive settlement and satisfy other site constraints.

Common types of deep foundations for bridges include driven piling, drilled shafts, micropiles and augercast piles. Driven piling is the most frequently-used type of deep foundation in Wisconsin. Drilled shafts may be advantageous where a very dense stratum must be penetrated to obtain required bearing, uplift or lateral resistance are concerns, or where obstructions may result in premature driving refusal or where piers need to be founded in areas of shallow bedrock or deep water. A drilled shaft may be more cost effective than driven piling when a drilled shaft is extended into a column and can be used to eliminate the need for a pile footing, pile casing or cofferdams.

Micropiles may be the best foundation alternatives where headroom is restricted or foundation retrofits are required at existing substructures. Micropiles tend to have a higher cost than traditional foundations.

Augercast piles are a potentially cost-effective foundation alternative, especially where lateral loads are minimal. However, restrictions on construction quality control including pile integrity and capacity need to be considered when augercast piles are being investigated. Augercast piles tend to have a higher cost than traditional foundations.

11.1.3 Cofferdams

At stream crossings, tremie-sealed cofferdams are frequently used when footing concrete is required to be placed below the surrounding water level.. The tremie-seal typically consists of a plain-cement concrete slab that is placed underwater (in the wet), within a closed-sided cofferdam that is generally constructed of sheetpiling. The seal concrete is placed after the excavation within the cofferdam has been completed to the proper elevation. The seal has three main functions: allowing the removal of water in the cofferdam so the footing concrete can be placed in the dry; serving as a counterweight to offset buoyancy due to differing water elevations within and outside of the cofferdam; and minimizing the possible deterioration of the excavation bottom due to piping and bottom heave. Concrete for tremie-seals is permitted to be placed with a tremie pipe underwater (in-the-wet). Footing concrete is typically required to be placed in-the-dry. In the event that footing concrete must be placed in-the-wet, a special provision for underwater inspection of the footing subgrade is required.

When bedrock is exposed in the bottom of any excavation and prior to placement of tremie concrete, the bedrock surface must be cleaned and inspected to assure removal of loose debris. This will assure good contact between the bedrock and eliminate the potential consolidation of loose material as the footing is loaded.

Cofferdams need to be designed to determine the required sheetpile embedment needed to provide lateral support, control piping and prevent bottom heave. The construction sequence must be considered to provide adequate temporary support, especially when each row of ring struts is installed. Over-excavation may be required to remove unacceptable materials at the base of the footing. Piles may be required within cofferdams to achieve adequate nominal bearing resistance. WisDOT has experienced a limited number of problems achieving adequate penetration of displacement piles within cofferdams when sheetpiling is excessively deep in granular material. Cofferdams are designed by the Contractor.

Refer to 13.11.5 for further guidance to determine the required thickness of cofferdam seals and to determine when combined seals and footings are acceptable.

11.1.4 Vibration Concerns

Vibration damage is a concern during construction, especially during pile driving operations. The selection process for the type of pile and hammer must consider the presence of surrounding structures that may be damaged due to high vibration levels. Pile driving operations can cause ground displacement, soil densification and other factors that can damage nearby buildings, structures and/or utilities. Whenever pile-driving operations pose the potential for damage to adjacent facilities (usually when they are located within approximately 100 feet), a vibration-monitoring program should be implemented. This program consists of requiring and reviewing a pile-driving plan submittal, conducting pre-driving and post-driving condition surveys and conducting the actual vibration monitoring with an approved seismograph. A special provision for implementing a vibration monitoring program is available and should be used on projects whenever pile-driving operations or other construction activities pose a potential threat to nearby facilities. Contact the geotechnical engineer for further discussion and assistance, if vibrations appear to be a concern.

11.2 Shallow Foundations

11.2.1 General

Design of a shallow foundation, also known as a spread footing, must provide adequate resistance against geotechnical and structural failure. The design must also limit deformations to within tolerable values. This is true for designs using ASD or LRFD. In many cases, a shallow foundation is the most economical foundation type, provided suitable soil conditions exist within a depth of approximately 0 to 15 feet below the base of the proposed foundation.

WisDOT policy item:

Design shallow foundations in accordance with the 4th Edition of the AASHTO *LRFD Bridge Design Specifications for Highway Bridges*. No additional guidance is available at this time.

Discussion is provided in 12.8 and 13.1 about design loads at abutments and piers, respectively. Live load surcharges at bridge abutments are described in 12.8.

11.2.2 Footing Design Considerations

The following design considerations apply to shallow foundations:

- Scour must not result in the loss of bearing or stability.
- Frost must not cause unacceptable movements.
- External or surcharge loads must be adequately supported.
- Deformation (settlement) and angular distortion must be within tolerable limits.
- Bearing resistance must be sufficient.
- Eccentricity requirements must be satisfied.
- Sliding resistance must be satisfied.
- Overall (global) stability must be satisfied.
- Uplift resistance must be sufficient.
- The effects of ground water must be mitigated and/or considered in the design.

11.2.2.1 Minimum Footing Depth

Foundation type selection and the preliminary design process require input from the geotechnical and hydraulic disciplines. The geotechnical engineer should provide guidance on the minimum embedment for shallow foundations that takes into consideration frost protection and the possible presence of unsuitable foundation materials. The hydraulic engineer should be consulted to assess scour potential and maximum scour depth for water crossings.

At shallow foundations bearing on rock, it is essential to obtain a proper connection to sound rock. Sometimes it is not possible to obtain deep footing embedment in granite or similar hard rock, due to the difficulty of rock removal.

11.2.2.1.1 Scour Vulnerability

Scour is a hydraulic erosion process caused by flowing water that lowers the grade of a water channel or riverbed. For this reason, scour vulnerability is an essential design consideration for shallow foundations. Scour can undermine shallow foundations or remove sufficient overburden to redistribute foundation forces, causing foundation displacement and detrimental stresses to structural elements. Excessive undermining of a shallow foundation leads to gross deformation and can lead to structure collapse.

Scour assessment will require streambed sampling and gradation analysis to define the median diameter of the bed material, D_{50}. Specific techniques for scour assessment, along with a detailed discussion of scour analysis and scour countermeasure design, are presented in the following publications:

- HEC 18 – *Evaluating Scour at Bridges*, 4th Edition
- HEC 20 – *Stream Stability at Highway Structures*, 3rd Edition
- HEC 23 – *Bridge Scour and Stream Instability Countermeasures - Experience, Selection and Design Guidance*, 2nd Edition

Foundations for new bridges and structures located within a stream or river should be located at an elevation below the maximum scour depth that is identified by the hydraulics engineer. In addition, the foundation should be designed deep enough such that scour protection is not required. If the maximum calculated scour depth elevation is below the practical limits for a shallow foundation, a deep foundation system should be used to support the structure.

11.2.2.1.2 Frost Protection

Shallow foundation footings must be embedded below the maximum depth of frost potential (frost depth) whenever frost heave is anticipated to occur in frost-susceptible soil and adequate moisture is available. This embedment is required to prevent foundation heave due to volumetric expansion of the foundation subgrade from freezing and/or to prevent settling due to loss of shear strength from thawing.

Frost susceptible material includes inorganic soil that contains at least 3 percent, dry weight, which is finer in size than 0.02 millimeters. Gravel that contains between 3 and 20 percent fines is least susceptible to frost heave. Bedrock is not considered frost susceptible if the bedrock formation is massive, dense and intact below the footing.

Foundation design is usually not governed by frost heave for foundations bearing on clean gravel and sand or very dense till. Frost heave is a concern whenever the water table, static or perched, is located within 5 feet of the freezing plane.

In Wisconsin, the maximum depth of frost potential generally ranges from approximately 4 feet in the southeastern part of the state to 6 feet in the northwestern corner of the state.

WisDOT policy item:

The minimum depth of embedment of shallow foundations shall be 4 feet, unless founded on competent bedrock.

Further discussion about frost protection in the design of bridge abutments and piers is presented in 12.5 and 13.6, respectively.

11.2.2.1.3 Unsuitable Ground Conditions

Footings should bear below weak, compressible or loose soil. In addition, some soil exhibits the potential for changes in volume due to the introduction or expulsion of water. These volumetric changes can be large enough to exceed the performance limits of a structure, even to the point of structural damage. Both expansive and collapsible soil is regional in occurrence. Neither soil type is well suited for shallow foundation support without a mitigation plan to address the potential of large soil volume changes in this soil, due to changes in moisture content. Expansive and collapsible soils seldom cause problems in Wisconsin.

It should be noted that the procedures presented herein for computing bearing resistance and settlement are applicable to naturally occurring soil and are not necessarily valid for conditions of modified ground such as uncontrolled fills, dumps, mines and waste areas. Due to the unpredictable behavior of shallow foundations in these types of random materials, deep foundations which penetrate through the random material, overexcavation to remove the random material, or subgrade improvement to improve material behavior is required at each substructure unit.

11.2.2.2 Tolerable Movement of Substructures Founded on Shallow foundations

The bridge designer shall account for any differential settlement (angular distortion) in the design.

WisDOT policy item:

For design of new bridge structures founded on shallow foundations, the maximum permissible movement is 1 inch of horizontal movement at the top of substructure units and 1.5 inches of total estimated settlement of each substructure unit at the Service Limit State.

The sequence of construction can be important when evaluating total settlement and angular distortion. The effects of embankment settlement, as well as settlement due to structure loads, should be considered when the magnitude of total settlement is estimated. It may be possible to manage the settlements after movements have stabilized, by monitoring movements and delaying critical structural connections such as closure pours or casting of decks that are continuous. Generally project timelines may restrict the time available for soil consolidation. Any project delays for geotechnical reasons must be thoroughly transmitted to, and analyzed by, design personnel.

11.2.2.3 Location of Ground Water Table

The location of the ground water table will impact both the stability and constructability of shallow foundations. A rise in the ground water table will cause a reduction in the effective vertical stress in soil below the footing and a subsequent reduction in the factored bearing resistance. A fluctuation in the ground water table is not usually a bearing concern at depths greater than 1.5 times the footing width below the bottom of footing.

WisDOT policy item:

The highest anticipated groundwater table should be used to determine the factored bearing resistance of footings. The Geotechnical Engineer should select this elevation based on the borings and knowledge of the specific site.

11.2.2.4 Sloping Ground Surface

The influence of a sloping ground surface must be considered for design of shallow foundations. The factored bearing resistance of the footing will be impacted when the horizontal distance is less than three times the footing width between the edge of sloping surface and edge of footing. Shallow foundations constructed in proximity to a sloping ground surface must be checked for overall stability. Procedures for incorporating sloping ground influence can be found in FHWA Publication SH-02-054, *Geotechnical Engineering Circular No. 6 Shallow Foundations* and LRFD 10.6.3.1.2c Considerations for Footings on Slopes.

11.2.3 Settlement Analysis

Settlement should be computed using Service I Limit State loads. Transient loads may be omitted to compute time-dependent consolidation settlement. Two aspects of settlement are important to structural designers: total settlement and differential settlement (ie relative displacement between adjacent substructure units). In addition to the amount of settlement, the designer also needs to determine the time rate for it to occur.

Vertical settlement can be a combination of elastic, primary consolidation and secondary compression movement. In general, the settlement of footings on cohesionless soil, very stiff to hard cohesive soil and rock with tight, unfilled joints will be elastic and will occur as load is applied. For footings on very soft to stiff cohesive soil, the potential for primary consolidation and secondary compression settlement components should be evaluated in addition to elastic settlement.

The design of shallow foundations on cohesionless soil (sand, gravel and non-plastic silt), either as found in-situ or as engineered fill, is often controlled by settlement potential rather than bearing resistance, or strength, considerations. The method used to estimate settlement of footings on cohesionless soil should therefore be reliable so that the predicted settlement is rarely less than the observed settlement, yet still reasonably accurate so that designs are efficient.

Elastic settlement is estimated using elastic theory and a value of elastic modulus based on the results of in-situ or laboratory testing. Elastic deformation occurs quickly and is usually small. Elastic deformation is typically neglected for movement that occurs prior to placement of girders and final bridge connections.

Semi-empirical methods are the predominant techniques used to estimate settlement of shallow foundations on cohesionless soil. These methods have been correlated to large databases of simple and inexpensive tests such as the Standard Penetration Test (SPT) and the Cone Penetrometer Test (CPT).

Consolidation of clays or clayey deposits may result in substantial settlement when the structure is founded on cohesive soil. Settlement may be instantaneous or may take weeks to years to complete. Furthermore, because soil properties may vary beneath the foundation, the duration of the consolidation and the amount of settlement may also vary with the location of the footing, resulting in differential settlement between footing locations. The consolidation characteristics of a given soil are a function of its past history. The reader is directed to FHWA Publication SA-02-054, *Geotechnical Engineering Circular No. 6 Shallow Foundations* for a detailed discussion on consolidation theory and principles.

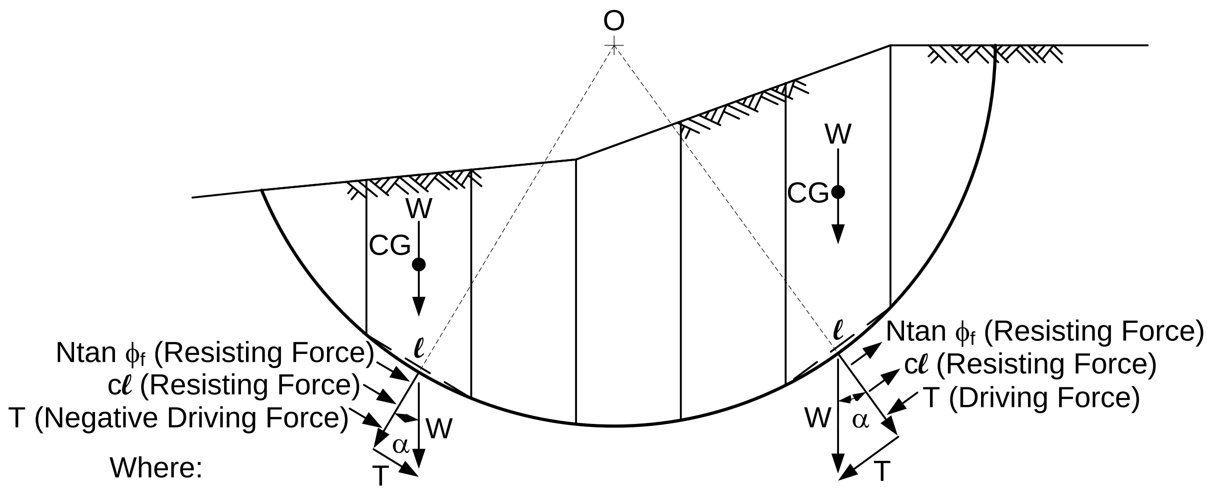
The rate of consolidation is usually of lesser concern for foundations, because superstructure damage will occur once the differential settlements become excessive. Shallow foundations are designed to withstand the settlement that will ultimately occur during the life of the structure, regardless of the time that it takes for the settlement to occur.

The design of footings bearing on intermediate geomaterials (IGM) or rock is generally controlled by considerations other than settlement. Intermediate geomaterial is defined as a material that is transitional between soil and rock in terms of strength and compressibility, such as residual soil, glacial till, or very weak rock. If a settlement estimate is necessary for shallow foundations supported on IGM or rock, a method based on elastic theory is generally the best approach. As with any of the methods for estimating settlement that use elastic theory, a major limitation is the engineer's ability to accurately estimate the modulus parameter(s) required by the method.

11.2.4 Overall Stability

Overall stability of shallow foundations that are located on or near slopes is evaluated using a limiting equilibrium slope stability analysis. Both circular arc and sliding-block type failures are considered using a Modified Bishop, simplified Janbu, Spencers or simplified wedge analysis, as applicable. The Service I load combination is used to analyze overall stability. A free body diagram for overall stability is presented in Figure 11.2-1.

Detailed guidance to complete a limiting equilibrium analysis is presented in FHWA Publication NHI-00-045, *Soils and Foundation Workshop Reference Manual* and **LRFD [11.6.2.3]**.



Where:

CG	=	Center of gravity
c	=	Cohesion along base of slice (psf)
ϕ_f	=	Angle of internal friction of drained soil (degrees)
W	=	Total weight of slice (lb/ft)
N	=	W cos α (lb/ft)
T	=	W sin α (lb/ft)
ℓ	=	length of slice (ft)

Figure 11.2-1

Free Body Diagram for Overall Stability

11.2.5 Footings on Engineered Fills

When shallow foundations are considered for placement on fill, further consideration is required. It is essential to satisfy the design tolerance with regard to total settlement, angular distortion and horizontal movement, including lateral squeeze of the embankment subgrade. The designer must consider the range of probable estimated movement and the impact that this range has on the overall structure performance. The anticipated movement of both new embankment fill and existing embankment materials must be assessed. If shallow foundations are considered, WisDOT requires a thorough subsurface investigation to evaluate settlement of the existing subgrade, including but not limited to continuous soil sampling. WisDOT does not typically place shallow foundations on general embankment fill. WisDOT may consider shallow foundations that are placed on engineered fill, such as that within MSE walls. WisDOT has placed a limited number of shallow foundations on MSE walls for single span bridges. Engineered fill typically consists of high-quality free-draining granular material that is not prone to behavior change due to moisture change, freeze-thaw action, long-term consolidation or shear failure. Engineered fill must also be tightly compacted. On occasion, engineered fill is used in combination with geotextile and/or geogrid to improve shear resistance and overall performance at approach embankments.

If it is not feasible to use a footing to support a sill abutment at the top of slope, it may be feasible to consider a shallow foundation at the bottom of abutment slope to support a full retaining abutment as discussed in 12.2. The increase in stem height will be offset by a reduction in required bridge span length.

11.2.6 Construction Considerations

Shallow foundations require field inspection during construction to confirm that the actual footing subgrade material is equivalent to, or better than, that considered for design. The prepared subgrade should be checked to confirm that the type and condition of the exposed subgrade will provide uniform bearing over the full length or width of footing. The exposed subgrade should be probed to identify possible underlying pockets of soft material that are covered by a thin crust of more competent material. Underlying pockets of soft material and unsuitable material should be over-excavated and replaced with competent material. The structural/geotechnical designer should be contacted if the revised field footing elevations vary by more than one foot lower or three feet higher than the plan elevations, due to differing conditions.

The exposed footing subgrade should be level and stepped, as needed. Stepped shallow foundations may be appropriate when the subsurface conditions vary over the length of substructure unit (footing). For simplicity, planned footing steps should be designated in maximum 4-foot increments. The number and spacing of footing steps is dependent on several factors including, but not limited to, site foundation conditions, temporary excavation support and dewatering requirements, frost and scour depth limitations, constructability, and construction sequence. In general, it is preferred to build uniform step-increments, to simplify construction. Typically the footing with the lowest elevation is constructed first to avoid excavation disturbance of other portions of the footing, as construction continues.

11.2.7 Geosynthetic Reinforced Soil (GRS) Abutment

Geosynthetic Reinforced Soil (GRS) abutments are a type of bridge foundation system typically supporting a single span precast superstructure.. The superstructure is supported on a course-grained soil (gravel) with layers of woven geotextile fabric spaced horizontally from the existing ground, to the base of the slab. The facing is a precast modular block and connected to the woven geotextile fabric. The following reference can be used for design, ‘Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, Publication Number: FHWA-HRT-11-026’

See 7.1.4.2 for guidance on GRS abutments.

11.3 Deep Foundations

When competent bearing soil is not present near the base of the proposed foundation, structure loads must be transferred to a deeper stratum by using deep foundations such as piles or drilled shafts (caissons). Deep foundations can be composed of piles, drilled shafts, micropiles or augered cast-in-place piles.

The primary functions of a deep foundation are:

- To transmit the load of the structure through a stratum of poor bearing capacity to one of adequate bearing capacity.
- To eliminate objectionable settlement.
- To transfer loads from a structure through erodible soil in a scour zone, to stable underlying strata.
- To anchor structures subjected to hydrostatic uplift or overturning forces.
- To resist lateral loads from earth pressures, as well as external forces.

11.3.1 Driven Piles

Deep foundation support systems have been in existence for many years. The first known pile foundations consisted of rows of timber stakes driven into the ground. Timber piles have been found in good condition after several centuries in a submerged environment. Several types of concrete piles were devised at the turn of the twentieth century. The earliest concrete piles were cast-in-place, followed by reinforced, precast and prestressed concrete piling. The requirement for longer piles with higher bearing capacity led to the use of concrete-filled steel pipe piles in about 1925. More recently, steel H-piles have also been specified due to ease of fabrication, higher bearing capacity, greater durability during driving and the ability to easily increase or decrease driven lengths.

11.3.1.1 Conditions Involving Short Pile Lengths

WisDOT policy generally requires piles to be driven a distance of 10 feet or greater below the original ground surface. Concern exists that short pile penetration in foundation materials of variable consistency may not adequately restrain lateral movements of substructure units. Pile penetrations of less than 10 feet are allowed if prebored at least 3 feet into solid rock. If conditions detailed in the Site Investigation Report clearly indicate that minimum pile penetration cannot be achieved, preboring should be included as a pay quantity. If there is a potential that preboring may not be necessary, do not include it in the plan documents. Piles which are not prebored into rock must not only meet the 10-foot minimum pile penetration criteria but must also have at least 5 feet of penetration through material with a blow count of at least 7 blows per foot. Piling should be “firmly seated” on rock after placement in prebored holes. The annular space between the cored holes in bedrock and piling should then be filled with concrete. Some sites may require casing during the preboring operation. If casing is required, it should be clearly indicated in the plan documents. Refer to 11.3.1.6 11.3.1.6 for additional information on preboring.

Foundations without piles (spread footings) should be given consideration at sites where pile penetrations of less than 10 feet are anticipated. The economics of the following two alternatives should be investigated:

1. Design for a shallow foundation founded at a depth where the foundation material is adequate. Embed the footing 6 inches into sound rock for lateral stability.

2. Excavate to an elevation where foundation material is adequate, and backfill to the bottom of footing elevation with suitable granular material or concrete.

If a substructure unit is located in a stream or lake, consideration should be given to the effects of the anticipated stream bed scour when selecting the footing type. Pile length computations should not incorporate pile resistance developed within the scour zone. The pile cross section should also be checked to ensure it can withstand the driving stresses necessary to penetrate through the anticipated scour depth and reach the required driving resistance plus the frictional resistance within the scour zone.

11.3.1.2 Pile Spacing

Arbitrary pile spacing rules specifying maximums and minimums are extensively used in foundation design. Proper spacing is dependent upon length, size, shape and surface texture of piles, as well as soil characteristics. A wide spacing of piles reduces heaving and possible uplifting of the pile, damage by tension due to heaving and the possibility of crushing from soil compression. Wider spacing more readily permits the tips of later-driven piles in the group to reach the same depths as the first piles and result in more even bearing and settlement. Large horizontal pressures are created when driving in relatively uncompressible strata, and damage may occur to piles already driven if piles are too closely spaced. In order to account for this, a minimum center-to-center spacing of 2.5 times the pile diameter is often required. LRFD [10.7.1.2] calls for a center-to-center pile spacing of not less than 2’-6” or 2.5 pile diameters (widths).

WisDOT policy item:

The minimum pile spacing is 2’-6” or 2.5 pile diameters, whichever is greater. For displacement piles located within cofferdams, or with estimated lengths ≥ 100 ft., the minimum pile spacing is 3.0 pile diameters. The minimum pile spacing for pile-encased piers and pile bents is 3’-0”. The maximum pile spacing is 8’-0” for abutments, pile encased piers, and pile bents, based on standard substructure designs.

See Chapter 13 – Piers for criteria on battered piles in cofferdams. The distance from the side of any pile to the nearest edge of footing shall not be less than 9”. Piles shall project at least 6” into the footings.

11.3.1.3 Battered Piles

Battered piles are used to resist large lateral loads or when there is insufficient lateral soil resistance within the initial 4 to 5 pile diameters of embedment. Battered piles are frequently used in combination with vertical piles. The lateral resistance of battered piling is a function of the vertical load applied to the pile group. Since the sum of the forces at the pile head must equal zero, increasing the number of battered piles does not necessarily increase the lateral load capacity of the pile group. Both the lateral passive resistance of the soil above the footing as well as the sliding resistance developed at the base of footing are generally neglected in design. See the standard details for further guidance when battered piles are used.

Piles are typically battered at 1 horizontal to 4 vertical. Hammer efficiencies must be reduced when piles are battered. Where negative skin friction loads are anticipated, battered piles should not be considered.

11.3.1.4 Corrosion Loss

Piling should be designed with sufficient corrosion resistance to assure a minimum design life of 75 years. Corrosive sites may included those with combinations of organic soils, high water table, man-made coal combustion products or waste materials, and those materials that allow air infiltration such as wood chips. Experience indicates that corrosion is not a practical problem for steel piles driven in natural soil, due primarily to the absence of oxygen in the soil. However, in fill material at or above the water table, moderate corrosion may occur and protection may be required. Concrete piles are prone to deterioration from exposure to excess concentrations of sulfate and/or chloride. Special consideration (including thicker pile shells, heavier pile sections, painting and concrete encasement) should be given to permanent steel piling that is used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see FDM Procedure 13-1-15). Typically, WisDOT does not increase pile sections or heavier pile sections to provide corrosion protection outside of these areas.

At potentially corrosive sites, encasement by cast-in-place concrete can provide the required protection for piles extending above the ground surface. All exposed piling should be painted. Additional guidance on corrosion is provided in **LRFD [10.7.5]**.

11.3.1.5 Pile Points

A study was conducted on the value of pile tips (pile points) on steel piles when driving into rock. The results indicated that there was very little penetration difference between the piles driven with pile points and those without. The primary advantages for specifying pile points are for penetrating or displacing boulders, driving through dense granular materials and hardpan layers, and to reduce the potential pile damage in hard driving conditions. Piling can generally be driven faster and in straighter alignment when points are used.

Conical pile points have also been used for round, steel piling (friction and point-bearing) in certain situations. These points can also be flush-welded if deemed necessary.

Standard details for pile points are available from the approved suppliers that are listed in WisDOT's current Product Acceptability List (PAL).

Pile points and preboring are sometimes confused. They are not interchangeable. Pile points can be used to help drive piles through soil that has gravel and/or cobbles or presents other difficult driving conditions. They can also be used to get a good ‘bite’ when ending piles on sloping bedrock surfaces. Points cannot be used to ensure that piles penetrate into competent bedrock. They may assist in driving through weathered zones of rock or soft rock, but will generally not be effective when penetration into hard rock is desired.

11.3.1.6 Preboring

If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. Preboring is required for displacement piles driven into new embankment that are over 10 feet in height. The WisDOT has developed special provisions to provide preboring requirements.

Except for point resistance piles, preboring should be terminated at least 5 feet above the scheduled pile tip elevation. When the pile is planned to be point resistance on rock, preboring may be advanced to plan pile tip elevation. Restrike is not performed when point-bearing piles are founded in rock within prebored holes. Preboring should only be used when appropriate, since many bridge contractors do not own the required construction equipment necessary for this work.

The annular space between the wall of the prebored hole and the pile is required to be backfilled. The annulus in bedrock should be filled with concrete or cement grout after the pile has been installed. Clean sand may be used to backfill the annulus over the depth of soil overburden. Backfill material should be deposited with either a tremie pipe or concrete pump to reduce potential arching (bridging) and assure that the complete depth of hole is filled.

11.3.1.7 Seating

Care must be taken when seating end bearing piles, especially when seating on bedrock with little to no weathered zone. When a pile is firmly seated on rock in prebored holes, pile driving to refusal is not required or recommended, to avoid driving overstress and pile damage. After reaching the predetermined prebore elevation, piles founded in soil are driven with a pile hammer to achieve the specified average penetration or set per blow for the final ten blows of driving.

11.3.1.8 Pile Embedment in Footings

The length of pile embedment in footings is determined based on the type and function of substructure unit and the magnitude of any uplift load.

WisDOT policy item:

Use a minimum 6-inch pile embedment in footings. This embedment depth is considered to result in a free (pinned) head connection for analysis. When the pile embedment depth into the footing is 2.0 feet or greater, the designer can assume a fixed head connection for analysis.

Additional pile embedment is required at some wing walls and at pile-encased substructures, especially where moment connections are required and where cofferdams are not used at stream crossings. Further guidance is provided in 13.6 and in the standard substructure details.

11.3.1.9 Pile-Supported Footing Depth

WisDOT policy item:

Place the bottom of pile-supported footings below the final ground surface at a minimum depth of 2.5 feet for sill abutments, 1.5 feet for sill abutments supported by MSE walls, and 4 feet for piers and other types of abutments.

11.3.1.10 Splices

Full-length piles should be used whenever practical. In no case should timber piles be spliced. Where splices are unavoidable, their number, location and details must be approved by WisDOT prior to pile splicing.

Splice details are shown on Wisconsin bridge plan standards for Pile Details. Splices are designed to develop the full strength of the pile section. Splices should be watertight for CIP concrete piles. Mechanical splice sleeves can be used to join sections of H-pile and pipe pile at greater depth where flexural resistance is not critical. Steel piling 20 feet or less in length is to be furnished in one unwelded piece. Piling from 20 to 50 feet in length can have two shop or field splices, and piling over 50 feet in length can be furnished with up to a maximum of four splices, unless otherwise stated in the project plan documents.

11.3.1.11 Painting

Normally, WisDOT paints all exposed sections of piling. This typically occurs at exposed pier bents.

11.3.1.12 Selection of Pile Types

The selection of a pile type for a given foundation application is made on the basis of soil type, stability under vertical and horizontal loading, long-term settlement, required method of pile installation, substructure type, cost comparison and estimated length of pile. Frequently more than one type of pile meets the physical and technical requirements for a given site. The performance of the entire structure controls the selection of the foundation. Primary considerations in choosing a pile type are the evaluation of the foundation materials and the selection of the substratum that provides the best foundation support.

Piling is generally used at piers where scour is possible, even though the streambed may provide adequate support without piling. In some cases, it is advisable to place footings at greater depths than minimum and specify a minimum pile penetration to guard against excessive scour beneath the footing and piling. Shaft resistance (skin friction) within the maximum depth of scour is assumed to be zero. When a large scour depth is estimated, this area of lost frictional support must be taken into account in the pile driving operations and capacities.

Subsurface conditions at the structure site also affect pile selection and details. The presence of artesian water pressure, soft compressible soil, cobbles and/or boulders, loose/firm uniform sands or deep water all influence the selection of the optimum type of pile for deep foundation support. For instance, WisDOT has experienced ‘running’ of displacement piling in certain areas that are composed of uniform, loose sands. The Department has also experienced difficulty driving displacement piles in denser sands within cofferdams, as consecutive piles are driven, due to compaction of the in-situ sand during pile installation within the cofferdam footprint.

If boulders or cobbles are anticipated within the estimated length of the pile, consideration should be given to increasing the cast-in-place (CIP) pile shell thickness to reduce the potential of pile damage due to high driving stresses. Other alternatives are to investigate the use of pile points or the use of HP piles at the site.

Environmental factors may be significant in the selection of the pile type. Environmental factors include areas subject to high corrosion, bacterial corrosion, abrasion due to moving debris or ice, wave action, deterioration due to cyclic wetting and drying, strong current and gradual erosion of riverbed due to scour. Concrete piles are susceptible to corrosion when exposed to alkaline soil or strong chemicals, especially in rivers and streams. Steel piles can suffer serious electrolysis deterioration if placed in an environment near stray electrical currents. Cast-in-place concrete piling is generally the preferred pile type on structure widenings where displacement piles are required. Timber pile is not to be used, even if timber pile was used on the original structure.

Displacement pile consisting of tapered steel is proprietary and can be an efficient type of friction pile for bearing in loose to medium-dense granular soil. Tapered friction piles may need to be installed with the aid of water jetting in dense granular soil. Straight-sided friction piles are recommended for placement in cohesive soils underlain by a granular stratum to develop the greatest combined shaft and point resistance. Steel HP or open-end pipe piles are best suited for driving through obstructions or fairly competent layers to bedrock. Foundations such as pier bents which may be subject to large lateral forces when located in deep and/or swiftly moving water require piles that can sustain large bending forces. Precast, prestressed concrete pile is best suited for high lateral loading conditions but is seldom used on Wisconsin transportation projects.

11.3.1.12.1 Timber Piles

Current design practice is not to use timber piles.

11.3.1.12.2 Concrete Piles

The three principal types of concrete pile are cast-in-place (CIP), precast reinforced and prestressed reinforced. CIP concrete pile types include piles cast in driven steel shells that remain in-place, and piles cast in unlined drilled holes or shafts. Driven-type concrete pile is discussed below in this section. Concrete pile cast in drilled holes is discussed later in this chapter and include drilled shafts (11.3.2), micropiles (11.3.3), and augered cast-in-place piles (11.3.4).

Depending on the type of concrete pile selected and the foundation conditions, the load-carrying capacity of the pile can be developed by shaft resistance, point resistance or a combination of both. Generally, driven concrete pile is employed as a displacement type pile.

When embedded in the earth, plain or reinforced concrete pile is generally not vulnerable to deterioration. The water table does not affect pile durability provided the concentration level is not excessive for acidity, alkalinity or chemical salt. Concrete pile that extends above the water surface is subject to abrasion damage from floating objects, ice debris and suspended solids. Deterioration can also result from frost action, particularly in the splash zone and from concrete spalling due to internal corrosion of the reinforcement steel. Generally, concrete spalls are a concern for reinforced concrete pile more than prestressed pile because of micro-cracks due to shrinkage, handling, placement and loading. Prestressing reduces crack width. Concrete durability increases with a corresponding reduction in aggregate porosity and water/cement ratio. WisDOT does not currently use prestressed reinforced concrete pile.

11.3.1.12.2.1 Driven Cast-In-Place Concrete Piles

Driven cast-in-place (CIP) concrete piles are formed by pouring concrete into a thin-walled closed-end steel shell which has been previously driven into the ground. A flat, oversize plate is typically welded to the bottom of the steel shell. Steel shells are driven either with or without a mandrel, depending on the wall thickness of the steel shell and the shell strength that is required to resist driving stress. Piling in Wisconsin is typically driven without the use of a mandrel. The minimum thickness of the steel shell should be that required for pile reinforcement and to resist driving stress. The Contractor may elect to furnish steel shells with greater thickness to permit their choice of driving equipment. A thin-walled shell must be carefully evaluated so that it does not collapse from soil pressure or deform from adjacent pile driving. Deformities or distortions in the pile shell could constrict the flow of concrete into the pile and produce voids or necking that reduce pile capacity. It is standard construction practice to inspect the open shell prior to concrete placement. Care must be exercised to avoid intermittent voids over the pile length during concrete placement.

Driven CIP concrete piles are considered a displacement-type pile, because the majority of the applied load is usually supported by shaft resistance. This pile type is frequently employed in slow flowing streams and areas requiring pile lengths of 50 to 120 feet. Driven CIP pile is generally selected over timber pile because of the availability of different diameters and wall thicknesses, the ability to adjust driven lengths and the ability to achieve greater resistances.

Driven CIP concrete piles may have a uniform cross section or may be tapered. The minimum cross-sectional area is required to be 100 and 50 square inches at the pile butt and tip, respectively. The Department has only used a limited number of tapered CIP piles and has experienced some driving problems with them.

For consistency with WisDOT design practice, the steel shell is ignored when computing the axial structural resistance of driven CIP concrete pile that is symmetrical about both principal axes. This nominal (ultimate) axial structural resistance capacity is computed using the following equation, neglecting the contribution of the steel shell to resist compression: **LRFD [Equation 5.7.4.4-3]**.

$$P_u \leq P_r = \phi P_n$$

Where:

$$P_n = 0.80(k_C \cdot f'_c \cdot (A_g - A_{st})) + f_y \cdot A_{st}$$

Where:

P_u	=	Factored axial force effect (kips)
P_r	=	Factored axial resistance without flexure (kips)
ϕ	=	Resistance factor
P_n	=	Nominal axial resistance without flexure (kips)
A_g	=	Gross area of concrete pile section (inches^2)
A_{st}	=	Total area of longitudinal reinforcement (inches^2)
k_C	=	Ratio of max. concrete compressive stress to specified compressive strength of concrete; k_C = 0.85 (for $f'_c \leq 10.0$ ksi)
f_y	=	Specified yield strength of reinforcement (ksi)
f'_c	=	Concrete compressive strength (ksi)

For cast-in-place concrete piles with steel shell and no steel reinforcement bars, A_{st} equals zero and the above equation reduces to the following.

$$P_n = 0.68 f'_c A_g$$

A resistance factor, ϕ, of 0.75 is used to compute the factored structural axial resistance capacity, as specified in **LRFD [5.5.4.2.1]**. For CIP piling there are no reinforcing ties, however the steel shell acts to confine concrete similar to ties.

$$P_r = 0.51 f_c A_g$$

For piles subject to large lateral loads, the structural pile capacity must also be checked for shear and combined stress against flexure and compression.

Piles subject to uplift must also be checked for tension resistance.

A concrete compressive strength of 4 ksi is the minimum value required by specification, while a value of 3.5 ksi is used in the structural design computations. Pile capacities are maximums, based on an assumed concrete compressive strength of 3.5 ksi. The concrete compressive strength of 3.5 ksi is based on construction difficulties and unknowns of placement. The

Geotechnical Site Investigation Report must be used as a guide in determining the nominal geotechnical resistance for the pile.

Any structural strength contribution associated with the steel shell is neglected in driven CIP concrete pile design. Therefore, environmentally corrosive sites do not affect driven CIP concrete pile designs. An exception is that CIP should not be used for exposed pile bents in corrosive environments as shown in the *Facilities Development Manual,* Procedure 13-1-15.

Based on the above equation, current WisDOT practice is to design driven cast-in-place concrete piles for factored (ultimate structural) axial compression resistances as shown in Table 11.3-5. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.** The minimum shell thickness is 0.219 inches for straight steel tube and 0.1793 inches for fluted steel shells, unless otherwise noted in the Geotechnical Site Investigation Report and stated in the project plans. Exposed piling (e.g. open pile bents) should not be less than 12 inches in diameter.

When cobbles or other difficult driving conditions are present, the minimum wall thickness for steel shells of driven cast-in-place concrete piles should be increased to 0.25 inches or thicker to facilitate driving without damaging the pile. A drivability analysis should be completed in design, to determine the required wall thickness based on site conditions and an assumed driving equipment.

Driven cast-in-place concrete pile is generally the most favorable displacement pile type since inspection of the steel shell is possible prior to concrete placement and more reliable control of concrete placement is attainable.

11.3.1.12.2.2 Precast Concrete Piles

Precast concrete pile can be divided into two primary types – reinforced concrete piles and prestressed concrete piles. These piles have parallel or tapered sides and are usually of rectangular or round cross section. Since the piles are usually cast in a horizontal position, the round cross section is not common because of the difficulty involved in filling a horizontal cylindrical form. Because of the somewhat variable subsurface conditions in Wisconsin and the need for variable length piles, these piles are currently not used in Wisconsin.

11.3.1.12.3 Steel Piles

Steel pile generally consist of either H-pile or pipe pile types. Both open-end and closed-end pipe pile are used. Pipe piles may be left open or filled with concrete, and can also have a structural shape or reinforcement steel inserted into the concrete. Open-end pipe pile can be socketed into bedrock with preboring.

Steel pile is typically top driven at the pile butt. However, closed-end pipe pile can also be bottom driven with a mandrel. Mandrels are generally not used in Wisconsin.

Steel pile can be used in friction, point-bearing, a combination of both, or rock-socketed piles. One advantage of steel pile is the ease of splicing or cutting to accommodate differing final constructed lengths.

Steel pile should not be used for exposed pile bents in corrosive environments as show in the *Facilities Development Manual, Procedure 13.1.15*.

The nominal (ultimate) axial structural compressive resistance of steel piles is designed in accordance with **LRFD [10.7.3.13.1]** as either non-composite or composite sections. Composite sections include concrete-filled pipe pile and steel pile that is encased in concrete. The nominal structural compressive resistance for non-composite and composite steel pile is further specified in **LRFD [6.9.4 and 6.9.5]**, respectively. The effective length of horizontally unsupported steel pile is determined in accordance with **LRFD [10.7.3.13.4]**. Resistance factors for the structural compression limit state are specified in **LRFD [6.5.4.2]**.

WisDOT policy item:

For steel H-piles, 50 ksi shall be used for pile design. For steel pipe piles, 35 ksi shall be used for pile design and drivability analyses. Plans shall note specified yield strength.

11.3.1.12.3.1 H-Piles

Steel piles are generally used for point-bearing piles and typically employ what is known as the HP-section (often called H-piles for brevity). Steel H-piles are rolled sections with wide flanges such that the depth of the section and the width of the flanges are approximately equal. The cross-sectional area and volume displacement are relatively small and as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered “non-displacement” piling.

H-piles are available in many sizes and lengths. Unspliced pile lengths up to 140 feet and spliced pile lengths up to 230 feet have been driven. Typical pile lengths range from 40 to 120 feet. Common H-pile sizes vary between 10 and 14 inches.

The current WisDOT practice is to design driven H-piles for the factored (ultimate structural) axial compression resistance as shown in Table 11.3-5. These values are based on $\phi_c = 0.5$ for severe driving conditions **LRFD [6.5.4.2]**. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.**

Since granular soil is largely incompressible, the principal action at the tip of the pile is lateral displacement of soil particles. Although it is an accepted fact that steel piles develop extremely high loads per pile when driven to point-bearing on rock, some misconceptions still remain that H-piles cannot function as friction piles. Load tests indicate that steel H-piles can function quite satisfactorily as friction piles in sand, sand-clay, silt-and-sand or hard clay. However, they are not as efficient as displacement piles in these conditions and typically drive to greater depths.

The surface area for pile frictional computations is considered to be the projected “box area” of the H-pile, and not the actual steel surface area.

Clay is compressible to a far greater degree than sand or gravel. As the solid particles are pressed into closer contact with each other and water is squeezed out of the voids, only small frictional resistance to driving is generated because of the lubricating action of the free water. However, after driving is completed, the lateral pressure against the pile increases due to dissipation of the pore water pressures. This causes the fine clay particles to increase adherence to the comparatively rough surface of the pile. Load is transferred from the pile to the soil by the resulting strong adhesive bond. In many types of clay, this bond is stronger than the shearing resistance of the soil.

In hard, stiff clays containing a low percentage of voids and pore water, the compressibility is small. As a result, the amount of displacement and compression required to develop the pile’s full capacity are correspondingly small. As an H-pile is driven into stiff clay, the soil trapped between the flanges and web usually becomes very hard due to the compression and is carried down with it. This trapped soil acts as a plug and the pile can also act as a displacement pile.

In cases where loose soil is encountered, considerably longer point-bearing steel piles are required to carry the same load as relatively short displacement-type piles. This is because a displacement-type pile, with its larger cross section, produces more compaction as it is driven through materials such as soft clays or loose organic silts. H-piles are not typically used in exposed pile bents due to concerns with debris catchment.

11.3.1.12.3.2 Pipe Piles

Pipe piles consist of seamless, welded or spiral welded steel pipes in diameters ranging from 7-3/4 to 24 inches. Other sizes are available, but they are not commonly used. Typical wall thicknesses range from 0.375-inch to 0.75-inch, with wall thicknesses of up to 1.5 inches possible. Pipe piles should be specified by grade with reference to ASTM A 252.

Pipe piles may be driven either open or closed end. If the end bearing capacity from the full pile toe area is required, the pile toe should be closed with a flat plate or a conical tip.

11.3.1.12.3.3 Oil Field Piles

The oil industry uses a very high quality pipe in their drilling operations. Every piece is tested for conformance to their standards. Oil field pipe is accepted as a point-bearing alternative to HP piling, provided the material in the pipe meets the requirements of ASTM A 252, Grade 3, with a minimum tensile strength of 120 ksi or a Brinell Hardness Number (BHN) of 240, a minimum outside diameter of 7-3/4 inches and a minimum wall thickness of 0.375-inch. The weight and area of the pipe shall be approximately the same as the HP piling it replaces. Sufficient bending strength shall be provided if the oil field pipe is replacing HP piling in a pile bent. Oil field pipe is driven open-ended and not filled with concrete. The availability of this pile type varies and is subject to changes in the oil industry.

11.3.1.12.4 Pile Bents

See 13.1 for criteria to use pile bents at stream crossings. When pile bents fail to meet these criteria, pile-encased pier bents should be considered. To improve debris flow, round piles are generally selected for exposed bents. Round or H-piles can be used for encased bents.

11.3.1.13 Tolerable Movement of Substructures Founded on Driven Piles

WisDOT policy item:

For design of new bridge structures founded on driven piles, limit the horizontal movement at the top of the foundation unit to 0.5 inch or less at the service limit state.

11.3.1.14 Resistance Factors

The nominal (ultimate) geotechnical resistance capacity of the pile should be based on the type, depth and condition of subsurface material and ground water conditions reported in the Geotechnical Site Investigation Report, as well as the method of analysis used to determine pile resistance. Resistance factors to compute the factored geotechnical resistance are presented in **LRFD [Table 10.5.5.2.3-1]** and are selected based on the method used to determine the nominal (ultimate) axial compression resistance. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal pile resistance. When construction controls, are used to improve the reliability of capacity prediction (such as pile driving analyzer or static load tests), the resistance factors used during final design should be increased in accordance with **LRFD [Table 10.5.5.2.3-1]** to reflect planned construction monitoring.

WisDOT exception to AASHTO:

WisDOT requires at least four (4) piles per group to support each substructure unit, including each column for multi-column bents. WisDOT does not reduce geotechnical resistance factors to satisfy redundancy requirements to determine axial pile resistance. Hence, redundancy resistance factors in **LRFD [10.5.5.2.3]** are not applicable to WisDOT structures. This exception applies to typical CIP concrete pile and H-pile foundations. Non-typical foundations (such as drilled shafts) shall be investigated individually.

No guidance regarding the structural design of non-redundant driven pile groups is currently included in *AASHTO LRFD*. Since WisDOT requires a minimum of 4 piles per substructure unit, structural design should be based on a load modifier, η, of 1.0. Further description of load modifiers is presented in **LRFD [1.3.4]**.

The following geotechnical resistance factors apply to the majority of the Wisconsin bridges that are founded on driven pile. On the majority of WisDOT projects, wave equation analysis and dynamic monitoring are not used to set driving criteria. This equates to typical resistance factors of 0.35 to 0.45 for pile design. A summary of resistance factors is presented in Table 11.3-1, which are generally used for geotechnical design on WisDOT projects.

Condition/Resistance Determination Method			Resistance Factor
Static Analysis – Used in Design Phase	Nominal Resistance of Single Pile in Axial Compression, φ_{stat}	Skin Friction and End Bearing in Clay and Mixed Soil Alpha Method	0.35
		Skin Friction and End Bearing in Sand Nordlund/Thurman Method	0.45
		Point Bearing in Rock	0.45
	Block Failure, φ_{bl}	Clay	0.60
	Uplift Resistance of Single Pile, φ_{up}	Clay and Mixed Soil Alpha Method	0.25
		Sand Nordlund Method	0.35
	Horizontal Resistance of Single Pile or Pile Group	All Soil Types and Rock	1.0
Nominal Resistance of Single Pile in Axial Compression – Dynamic Analysis – for the Hammer and Pile Driving System Actually - used During Construction for Pile Installation, φ_{dyn}		FHWA-modified Gates dynamic pile driving formula (end of drive condition only)	0.50
		Wave equation analysis, without pile dynamic measurements or load test, at end of drive condition only	0.50
		Driving criteria established by dynamic test with signal matching at beginning of redrive conditions only of at least one production pile per substructure, but no less than the number of tests per site provided in **LRFD [Table 10.5.5.2.3-3]**; quality control of remaining piles by calibrated wave equation and/or dynamic testing	0.65

Table 11.3-1
Geotechnical Resistance Factors for Driven Pile

Resistance factors for structural design of piles are based on the material used, and are presented in the following sections of *AASHTO LRFD*:

- Concrete piles – **LRFD [5.5.4.2.1]**
- Steel piles – **LRFD [6.5.4.2]**

11.3.1.15 Bearing Resistance

A pile foundation transfers load into the underlying strata by either shaft resistance, point resistance or a combination of both. Any driven pile will develop some amount of both shaft and point resistance. However, a pile that receives the majority of its support capacity by friction or adhesion from the soil along its shaft is referred to as a friction pile, whereas a pile that receives the majority of its support from the resistance of the soil near its tip is a point resistance (end bearing) pile.

The design pile capacity is the maximum load the pile can support without exceeding the allowable movement criteria. When considering design capacity, one of two items may govern the design – the nominal (ultimate) geotechnical resistance capacity or the structural resistance capacity of the pile section. This section focuses primarily on the geotechnical resistance capacity of a pile.

The factored load that is applied to a single pile is carried jointly by the soil beneath the tip of the pile and by the soil around the shaft. The total factored load is not permitted to exceed the factored resistance of the pile foundation for each limit state in accordance with **LRFD [1.3.2.1 and 10.7.3.8.6]**. The factored bearing resistance, or pile capacity, of a pile is computed as follows:

$$\sum \eta_i \gamma_i Q_i \leq R_r = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s$$

Where:

η_i	=	Load modifier
γ_i	=	Load factor
Q_i	=	Force effect (tons)
R_r	=	Factored bearing resistance of pile (tons)
R_n	=	Nominal resistance (tons)
R_p	=	Nominal point resistance of pile (tons)
R_s	=	Nominal shaft resistance of pile (tons)
φ	=	Resistance factor
φ_{stat}	=	Resistance factor for driven pile, static analysis method

This equation is illustrated in Figure 11.3-1.

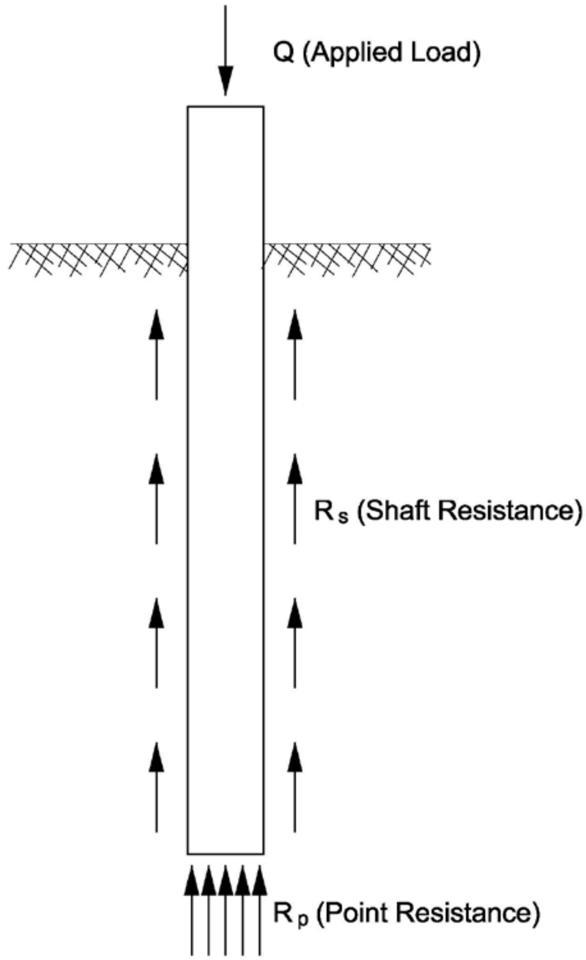


Figure 11.3-1
Resistance Distribution for Axially-Loaded Pile

11.3.1.15.1 Shaft Resistance

The shaft resistance of a pile is estimated by summing the frictional resistance developed in each of the different soil strata.

For non-cohesive (granular) soil, the total shaft resistance can be calculated using the following equation (based on the Nordlund/Thurman Method):

$$R_s = \Sigma C_d D K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos(\omega)}$$

Where:

R_s = Total shaft resistance capacity (tons)

C_d = Pile perimeter (feet)

D = Pile segment length (feet)

K_δ = Coefficient of lateral earth pressure at mid-point of soil layer under consideration from **LRFD [Figures 10.7.3.8.6f-1 through 10.7.3.8.6f-4]**

C_F = Correction factor for K_δ when $\delta \neq \phi_f$, from **LRFD [Figure 10.7.3.8.6f-5],** whereby ϕ_f = angle of internal friction for drained soil

σ_v' = Effective overburden pressure at midpoint of soil layer under consideration (tsf)

δ = Friction angle between the pile and soil obtained from **LRFD [Figure 10.7.3.8.6f-6]** (degrees)

ω = Angle of pile taper from vertical (degrees)

For cohesive (fine-grained) soil, the total shaft resistance can be calculated using the following equation (based on the alpha method):

$$R_s = \Sigma \alpha S_u C_d D$$

Where:

R_s = Total (nominal) shaft resistance capacity (tons)

α = Adhesion factor based on the undrained shear strength from **LRFD [Figure 10.7.3.8.6b-1]**

S_u = Undrained shear strength (tsf)

C_d = Pile perimeter (feet)

D = Pile segment length (feet)

Typical values of nominal shaft resistance for various soils are presented in Table 11.3-2 and Table 11.3-3. The values presented are average ranges and are intended to provide orders of magnitude only. Other conditions such as layering sequences, drilling information, ground water, thixotropy and clay sensitivity must be evaluated by experienced geotechnical engineers and analyzed using principles of soil mechanics.

Soil Type	q_u(1) (tsf)	Nominal Shaft Resistance (psf)
Very soft clay	0 to 0.25	---
Soft clay	0.25 to 0.5	200 to 450
Medium clay	0.5 to 1.0	450 to 800
Stiff clay	1.0 to 2.0	800 to 1,500
Very stiff clay	2.0 to 4.0	1,500 to 2,500
Hard clay	4.0	2,500 to 3,500
Silt	---	100 to 400
Silty clay	---	400 to 700
Sandy clay	---	400 to 700
Sandy silt	---	600 to 1,000
Dense silty clay	---	900 to 1,500

(1) Unconfined Compression Strength

Table 11.3-2
Typical Nominal Shaft Resistance Values for Cohesive Material

Soil Type	N_{160}(1)	Nominal Shaft Resistance (psf)
Very loose sand and silt or clay	0 to 6	50 to 150
Medium sand and silt or clay	6 to 30	400 to 600
Dense sand and silt or clay	30 to 50	600 to 800
Very dense sand and silt or clay	over 50	800 to 1,000
Very loose sand	0 to 4	700 to 1,700
Loose sand	4 to 10	700 to 1,700
Firm sand	10 to 30	700 to 1,700
Dense sand	30 to 50	700 to 1,700
Very dense sand	over 50	700 to 1,700
Sand and gravel	---	1,000 to 3,000
Gravel	---	1,500 to 3,500

(1) Standard Penetration Value (AASHTO T206) corrected for both overburden and hammer efficiency effects (blows per foot).

Table 11.3-3

Typical Nominal Shaft Resistance Values for Granular Material

Shaft resistance values are dependent upon soil texture, overburden pressure and soil cohesion but tend to increase with depth. However, experience in Wisconsin has shown that shaft resistance values in non-cohesive materials reach constant final values at depths of 15 to 25 pile diameters in loose sands and 25 to 35 pile diameters in firm sands.

In computing shaft resistance, the method of installation must be considered as well as the soil type. The method of installation significantly affects the degree of soil disturbance, the lateral stress acting on the pile, the friction angle and the area of contact. Shafts of prebored piles do not always fully contact the soil; therefore, the effective contact area is less than the shaft surface area. Driving a pile in granular material densifies the soil and increases the friction angle. Driving also displaces the soil laterally and increases the horizontal stress acting on the pile. Disturbance of clay soil from driving can break down soil structure and increase pore pressures, which greatly decreases soil strength. However, some or all of the strength recovers following reconsolidation of the soil due to a decrease in excess pore pressure over time. Use the initial soil strength values for design purposes. The type and shape of a pile also affects the amount of shaft resistance developed, as described in 11.3.1.12.

11.3.1.15.2 Point Resistance

The point resistance, or end bearing capacity, of a pile is estimated from modifications to the bearing capacity formulas developed for shallow footings.

For non-cohesive soils, point resistance can be calculated using the following equation (based on the Nordlund/Thurman Method):

$$R_p = A_p\,\alpha_t\,N'_q\,\sigma_v' \leq q_L\,A_p$$

Where:

R_p	=	Point resistance capacity (tons)
A_p	=	Pile end area (feet^2)
α_t	=	Dimensionless factor dependent on depth-width relationship from **LRFD [Figure 10.7.3.8.6f-7]**
N'_q	=	Bearing capacity factor from **LRFD [Figure 10.7.3.8.6f-8]**
σ_v'	=	Effective overburden pressure at the pile point $\leq$ 1.6 (tsf)
q_L	=	Limiting unit point resistance from **LRFD [Figure 10.7.3.8.6f-9]** (tsf)

For cohesive soils, point resistance can be calculated using the following equation:

$$R_p = 9 S_u A_p$$

Where:

R_p = Point resistance capacity (tons)

S_u = Undrained shear strength of the cohesive soil near the pile base (tsf)

A_p = Pile end area (feet^2)

This equation represents the maximum value of point resistance for cohesive soil. This value is often assumed to be zero because substantial movement of the pile tip (1/10 of the pile diameter) is needed to mobilize point resistance capacity. This amount of tip movement seldom occurs after installation.

A point resistance (or end bearing) pile surrounded by soil is not a structural member like a column. Both experience and theory demonstrate that there is no danger of a point resistance pile buckling due to inadequate lateral support if it is surrounded by even the very softest soil. Therefore, pile stresses can exceed column stresses. Although, exposed pile bent piles may act as structural columns.

11.3.1.15.3 Group Capacity

The nominal resistance capacity of pile groups may be less than the sum of the individual nominal resistances of each pile in the group for friction piles founded in cohesive soil. For pile groups founded in cohesive soil, the pile group must be analyzed as an equivalent pier for block failure in accordance with **LRFD [10.7.3.9]**. WisDOT no longer accepts the Converse-Labarre method of analysis to account for group action. If the pile group is tipped in a firm stratum overlying a weak layer, the weak layer should be checked for possible punching failure in accordance with **LRFD [10.6.3.1.2a]**. Experience in Wisconsin indicates that in most thixotropic clays where piles are driven to a hammer bearing as determined by dynamic formulas, pile group action is not the controlling factor to determine pile resistance capacity. For pile groups in sand, the sum of the nominal resistance of the individual piles always controls the group resistance.

11.3.1.16 Lateral Load Resistance

Structures supported by single piles or pile groups are frequently subjected to lateral forces from lateral earth pressure, live load forces, wave action, ice loads and wind forces. Piles subjected to lateral forces must be designed to meet combined stress and deflection criteria to prevent impairment or premature failure of the foundation or superstructure. To solve the soil-structure interaction problems, the designer must consider the following:

- Pile group configuration.

- Pile stiffness.
- Degree of fixity at the pile connection with the pile footing.
- Maximum bending moment induced on the pile from the superstructure load and moment distribution along the pile length.
- Probable points of fixity near the pile tip.
- Soil response (P-y method) for both the strength and service limit states.
- Pile deflection permitted by the superstructure at the service limit state.

If a more detailed lateral load investigation is desired, a P-y analysis is typically performed using commercially available software such as COM624P, FB Multi-Pier or L-Pile. A resistance factor of 1.0 is applied to the soil response when performing a P-y analysis using factored loads since the soil response represents a nominal (ultimate) condition. For a more detailed analysis of lateral loads and displacements, refer to the listed FHWA design references at the end of this chapter or a geotechnical engineering book.

WisDOT policy item:

A detailed analysis is required for the lateral resistance of piles used in A3 and A4 abutments.

11.3.1.17 Other Design Considerations

Several other topics should be considered during design, as presented below.

11.3.1.17.1 Downdrag Load

Negative shaft resistance (downdrag) results in the soil adhesion forces pulling down the pile instead of the soil adhesion forces resisting the applied load. This can occur when settlement of the soil through which the piling is driven takes place. It has been found that only a small amount of settlement is necessary to mobilize these additional pile (drag) loads. This settlement occurs due to consolidation of softer soil strata caused by such items as increased embankment loads (due to earth fill) or a lowering of the existing ground water elevation. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer acting to produce negative skin resistance. When this condition is present, the designer may provide time to allow consolidation to occur before driving piling, or **LRFD [10.7.3.8.6]** may be used to estimate the available pile resistance to withstand the downdrag plus structure loads. Other alternatives are to pre-auger the piling, drive the pile to bearing within a permanent pipe sleeve that is placed from the base of the substructure unit to the bottom of the soft soil layer(s), coat the pile with bitumen above the compressible soil strata or use proprietary materials to encase the piles (within fill constructed after the piling is installed). The Department has experienced problems with bitumen coatings.

The factored axial compression resistance values given for H-piles in Table 11.3-5 are conservative and based on Departmental experience to avoid overstressing during driving. For H-piles in end bearing, loading from downdrag is allowed in addition to the normal pile loading, since this is a post-driving load. Use the values given in Table 11.3-5 and design piling as usual. Additionally, up to 45, 60, and 105 tons downdrag for HP 10x42, HP 12x53, and HP 14x73 piles respectively is allowed when the required driving resistance is determined by the modified Gates dynamic formula.

11.3.1.17.2 Lateral Squeeze

Lateral squeeze as described in **LRFD [10.7.2.6]** occurs when pile supported abutments are constructed on embankments and/or MSE walls over soft soils. Typically, the piles are installed prior to completion of the embankment and/or MSE wall, and therefore are potentially subject to subsurface soil instability. If the embankment and/or MSE wall has a marginal factor of safety with regards to slope stability, then lateral squeeze has the potential to laterally deflect the piles and tilt the abutment. Typically, if the shear strength of the subsurface soil is less than the height of the embankment times the unit weight of the embankment divided by three, then damage from lateral squeeze could be expected.

If this is a potential problem, the following are the recommended solutions from the *FHWA Design and Construction of Driven Piles Manual*:

1. Delay installation of abutment piling until after settlement has stabilized (best solution).

2. Provide expansion shoes large enough to accommodate the movement.

3. Use steel H-piles strong enough and rigid enough to provide both adequate strength and deflection control.

4. Use lightweight fill to reduce driving forces.

11.3.1.17.3 Uplift Resistance

Uplift forces may also be present, both permanently and intermittently, on a pile system. Such forces may occur from hydrostatic uplift or cofferdam seals, ice uplift resulting from ice grip on piles and rising water, wind uplift due to pressures against high structures or frost uplift. In the absence of pulling test data, the calculated factored shaft resistance should be used to determine static uplift capacity to demand ratio (CDR). A minimum CDR value of 1.0 is required. Generally, the type of pile with the largest perimeter is the most efficient in resisting uplift forces.

11.3.1.17.4 Pile Setup and Relaxation

The nominal resistance of a deep foundation may change over time, particularly for driven piles. The nominal resistance may increase (setup) during dissipation of excess pore pressure, which developed during pile driving, as soil particles reconsolidate after the soil has been remolded during driving. The shaft resistance may decrease (relaxation) during dissipation of negative pore pressure, which was induced by physical displacement of soil during driving. If the potential for soil relaxation is significant, a non-displacement pile is preferred over a displacement type pile. Relaxation may also occur as a result of a deterioration of the bearing stratum following driving-induced fracturing, especially for point-bearing piles founded on non-durable bedrock. Relaxation is generally associated with densely compacted granular material.

Pile setup has been found to occur in some fine-grained soil in Wisconsin. Pile setup should not be included in pile design unless pre-construction load tests are conducted to determine site-specific setup parameters. The benefits of obtaining site-specific setup parameters could include shortening friction piles and reducing the overall foundation cost. Pile driving resistance would need to be determined at the end of driving and again later after pore pressure dissipation. Restrike tests involve additional taps on a pile after the pile has been driven and a waiting period (generally 24 to 72 hours) has elapsed. The dynamic monitoring analysis are used to predict resistance capacity and distribution over the pile length.

CAPWAP(CAse Pile Wave Analysis Program) is a signal matching software. CAPWAP uses dynamic pile force and velocity data to discern static and dynamic soil resistance, and then estimate static shaft and point resistance for driven pile. Pile top force and velocity are calculated based on strain and acceleration measurements during pile driving, with a pile driving analyzer (PDA). CAPWAP is based on the wave equation model which characterizes the pile as a series of elastic beam elements, and the surrounding soil as plastic elements with damping (dynamic resistance) and stiffness (static resistance) properties.

Typically, a test boring is drilled and a static load test is performed at test piles where pile setup properties are to be determined. Typical special provisions have been developed for use on projects incorporating aspects of pile setup. Pile setup is discussed in greater detail in FHWA Publication NHI-05-042, *Design and Construction of Driven Pile Foundations*.

Restrike tests with an impact hammer can be used to identify change in pile resistance due to pile setup or relaxation. Restrike is typically performed by measuring pile penetration during the first 10 blows by a warm hammer. Due to setup, it is possible that the hammer used for initial driving may not be adequate to induce pile penetration and a larger hammer may be required to impart sufficient energy for restrike tests. Only warm hammers should be used for restrikes by first applying at least 20 blows to another pile.

Restrike tests with an impact hammer must be used to substantiate the resistance capacity and integrity of pile that is initially driven with a vibratory hammer. Vibratory hammers may be used with approval of the engineer. Other than restrikes with an impact hammer, no formula exists to reliably predict the resistance capacity of a friction pile that is driven with a vibratory hammer.

11.3.1.17.5 Drivability Analysis

In order for a driven pile to develop its design geotechnical resistance, it must be driven into the ground without damage. Stresses developed during driving often exceed those developed under even the most extreme loading conditions. The critical driving stress may be either compression, as in the case of a steel H-pile, or tension, as in the case of a concrete pile.

Drivability is treated as a strength limit state. The geotechnical engineer will perform the evaluation of this limit state during design based on a preliminary dynamic analysis using wave equation techniques. These techniques are used to document that the assumed pile driving hammers are capable of mobilizing the required nominal (ultimate) resistance of the pile at driving stress levels less than the factored driving resistance of the pile. Drivability can often be the controlling strength limit state check for a pile foundation. This is especially true for high capacity piles driven to refusal on rock.

Drivability analysis is required by **LRFD [10.7.8]**. A drivability evaluation is needed because the highest pile stresses are usually developed during driving to facilitate penetration of the pile to the required resistance. However, the high strain rate and temporary nature of the loading during pile driving allow a substantially higher stress level to be used during installation than for service. The drivability of candidate pile-hammer-system combinations can be evaluated using wave equation analyses.

As stated in the 2004 FHWA Design and Construction of Driven Pile Foundations Manual:

> "The wave equation does not determine the capacity of the pile based on soil boring data. The wave equation calculates a penetration resistance for an assumed ultimate capacity, or conversely it assigns estimated ultimate capacity to a pile based upon a field observed penetration resistance."
>
> "The accuracy of the wave equation analysis will be poor when either soil model or soil parameters inaccurately reflect the actual soil behavior, and when the driving system parameters do not represent the state of maintenance of hammer or cushions."

The following presents potential sources of wave equation errors.

- Hammer Data Input, Diesel Hammers
- Cushion Input
- Soil Parameter Selection

LRFD [C10.7.8] states that the local pile driving results from previous drivability analyses and historical pile driving experience can be used to refine current drivability analyses. WisDOT recommends using previous pile driving records and experience when performing and evaluating drivability analyses. These correlations with past pile driving experience allow modifications of the input values used in the drivability analysis, so that results agree with past construction findings.

Driving stress criteria are specified in the individual LRFD material design sections and include limitations of unfactored driving stresses in piles based on the following:

- Yield strength in steel piles, as specified in **LRFD [6.4.1]**
- Ultimate compressive strength of the gross concrete section, accounting for the effective prestress after losses for prestressed concrete piles loaded in tension or compression, as specified in **LRFD [5.7.4.4]**

Though there are a number of ways to assess the drivability of a pile, the steps necessary to perform a drivability analysis are typically as follows:

1. Estimate the total resistance of all soil layers. This may include layers that are not counted on to support the completed pile due to scour or potential downdrag, but will have to be driven through. WisDOT recommends using the values for quake and damping provided in the FHWA Design and Construction of Driven Pile Foundations Manual.

   In addition, the soil resistance parameters should be reduced by an appropriate value to account for the loss of soil strength during driving. The following table provides some guidelines based on Table 9-19 of the FHWA Design and Construction of Driven Pile Foundations Manual:

Soil Type	Recommended Soil Set Up Factor[1]	Percentage Loss of Soil Strength during Driving
Clay	2.0	50 percent
Silt – Clay	1.5[2]	33 percent
Silt	1.5	33 percent
Sand – Clay	1.5	33 percent
Sand – Silt	1.2	17 percent
Fine Sand	1.2	17 percent
Sand	1.0	0 percent
Sand - Gravel	1.0	0 percent
Notes: 1. Confirmation with local experience recommended 2. The value of 1.5 is higher than the FHWA Table 9-19 value of 1.0 based upon WisDOT experience.		

Table 11.3-4
Soil Resistance Factors

Incorporation of loss of soil strength and soil set-up should only be accounted for in the pile drivability analyses. Typically, WisDOT does not include set-up in static pile design analyses.

2. Select a readily available hammer. The following hammers have been used by Wisconsin Bridge Contractors: Delmag D-12-42, Delmag D-12-32, Delmag D-12, Delmag D-15, Delmag D-16-32, Delmag D-19, Delmag D-19-32, Delmag D-19-42, Delmag D-25, Delmag D-30-32, Delmag D-30, Delmag D-36, MKT-7, Kobe K-13, Gravity Hammer 5K.

3. Model the driving system, soil and pile using a wave equation program. The driving system generally includes the pile-driving hammer, and elements that are placed between the hammer and the top of pile, which include the helmet, hammer cushion, and pile cushion (concrete piles only). Pile splices are also modeled. Compute the driving stress using the drivability option for the wave equation, which shows the pile compressive stress an blow counts versus depth for the given soil profile.

4. Determine the permissible driving stress in the pile. During the design stage, it is often desirable to select a lower driving stress than the maximum permitted. This will allow the contractors greater flexibility in hammer selection. WisDOT generally limits driving stress to 90 percent of the steel yield strength

5. Evaluate the results of the drivability analysis to determine a reasonable blow count (that is, ranges from 25 blows per foot to 120 blows per foot) associated with the permissible driving stress.

The goal of the drivability study is to evaluate the potential for excessive driving stresses and to determine that the pile/soil system during driving will result in reasonable blow counts. The drivability study is not intended to evaluate the ultimate pile capacity or establish plan lengths. If the wave equation is used to set driving criteria, then contact the Bureau of Technical Services, Geotechnical Engineering Unit to discuss the proper procedures.

11.3.1.17.6 Scour

During design, estimated pile lengths are increased to compensate for scour loss. The scour depth is estimated and used to compute the estimated shaft resistance that is lost over the scour depth (exposed pile length). The required pile length is then increased to compensate for the resistance capacity that is lost due to scour. The pile length is increased based on the following equation:

$$R_n = R_{n\text{-stat}} + R_{n\text{-scour}}$$

Where:

R_n	=	Nominal shaft resistance capacity, adjusted for scour effect (tons)
$R_{n\text{-stat}}$	=	Nominal shaft resistance based on static analysis, without scour consideration (tons)
$R_{n\text{-scour}}$	=	Nominal shaft resistance lost (negative value) over the exposed pile length due to scour (tons)

WisDOT policy item:

If there is potential for scour at a site, account for the loss of pile resistance from the material within the scour depth. The designer must not include any resistance provided by this material when determining the nominal pile resistance. Since the material within the scour depth may be present during pile driving operations, the additional resistance provided by this material shall be included when determining the required driving resistance. The designer should also consider minimum pile tip elevation requirements.

11.3.1.17.7 Typical Pile Resistance Values

Table 11.3-5 shows the typical pile resistance values for several pile types utilized by the Department. The table shows the Nominal Axial Compression Resistance (Pn), which is a function of the pile materials, the Factored Axial Compression Resistance (Pr), which is a function of the construction procedures, and the Required Driving Resistance, which is a function of the method used to measure pile capacity during installation. The bridge designer uses the Factored Axial Compression Resistance to determine the number and spacing of the piles. The Required Driving Resistance is placed on the plans. See 6.3.2.1-7 for details regarding plan notes.

Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Resistance (Pn) (tons) (2)(3)(6)	(ϕ)	Maximum Factored Resistance (Pr) (tons) (4)	Modified Gates Driving Criteria: Factored Resistance (Pr) (ϕ = 0.50) (tons)	Modified Gates Driving Criteria: Required Driving Resistance (Rn_{dyn}) (tons) (5)	PDA/CAPWAP Driving Criteria: Factored Resistance (Pr) (ϕ = 0.65) (tons)	PDA/CAPWAP Driving Criteria: Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles									
10 ¾“	0.219	83.5	99.4	0.75	75	55 (8)	110	72 (8)	110
10 ¾“	0.250	82.5	98.2	0.75	74	65 (8)	130	75 (9)	115
10 ¾“	0.365	78.9	93.8	0.75	70	75 (9)	150	75 (9)	115
10 ¾“	0.500	74.7	88.8	0.75	67	75 (9)	150	75 (9)	115
12 ¾”	0.250	118.0	140.4	0.75	105	80 (8)	160	104 (8)	160
12 ¾”	0.375	113.1	134.6	0.75	101	105 (9)	210	104 (9)	160
12 ¾”	0.500	108.4	129.0	0.75	97	105 (9)	210	104 (9)	160
14”	0.250	143.1	170.3	0.75	128	85 (8)	170	111 (8)	170
14”	0.375	137.9	164.1	0.75	123	120 (8)	240	120	185
14”	0.500	132.7	158.0	0.75	118	120 (9)	240	120 (9)	185
16”	0.375	182.6	217.3	0.75	163	145 (8)	290	159	245
16”	0.500	176.7	210.3	0.75	158	160 (9)	320	159 (9)	245
H-Piles									
10 x 42	NA(1)	12.4	310.0	0.50	155	90	180 (10)	117	180 (10)
12 x 53	NA(1)	15.5	387.5	0.50	194	110	220 (10)	143	220 (10)
14 x 73	NA(1)	21.4	535.0	0.50	268	125	250 (10)	162	250 (10)

Table 11.3-5
Typical Pile Axial Compression Resistance Values

Notes:

1. NA – not applicable

2. For CIP Piles: Pn = 0.8 (k_C * f'c * Ag + fy * As) **LRFD [5.7.4.4-3]**. k_C = 0.85 (for $f'_C \leq$ 10.0 ksi). Neglecting the steel shell, equation reduces to 0.68 * f'c * Ag.

   f'c = compressive strength of concrete = 3,500 psi

3. For H-Piles: Pn = (0.66^{λ} * Fe * As) **LRFD [6.9.5.1-1]** (λ = 0 for piles embedded in the ground below the substructure, i.e. no unsupported lengths)

Fe = fy = yield strength of steel = 50,000 psi

4. Pr = ϕ * Pn

   ϕ = 0.75 (**LRFD [5.5.4.2.1]** for axial compression concrete)

   ϕ = 0.50 (**LRFD [6.5.4.2]** for axial steel, for difficult driving conditions)

5. The Required Driving Resistance is the lesser of the following:

   - Rn_{dyn} = Pr / φ_{dyn}

     φ_{dyn} = 0.50 for construction driving criteria using modified Gates dynamic formula

     φ_{dyn} = 0.65 for construction driving criteria using PDA/CAPWAP

   - The maximum allowable driving stress based on 90 percent of the specified yield stress = 35,000 psi for CIP piles and 50,000 psi for H-Piles (see note 10).

6. Values for Axial Compression Resistance are calculated assuming the pile is fully supported. Piling not in the ground acts as an unbraced column. Calculations verify that the pile values given in Table 11.3-5 are valid for open pile bents within the limitations described in 13.2.2. Cases of excessive scour require the piling to be analyzed as unbraced columns above the point of streambed fixity.

7. If less than the maximum axial resistance, P_r, is required by design, state only the required corresponding driving resistance on the plans.

8. The Factored Axial Compression Resistance is controlled by the maximum allowable driving resistance based on 90 percent of the specified yield stress of steel rather than concrete capacity.

9. Values were rounded up to the value above so as to not penalize the capacity of the thicker walled pile of the same diameter. (Wisconsin is conservative in not considering the pile shell in the calculation of the Factored Axial Compression Resistance. Rounded values utilize some pile shell capacity)

10. Rn_{dyn} values given for H-Piles are representative of past Departmental experience (rather than Pn x ø) and are used to avoid problems associated with overstressing during driving. These Rn_{dyn} values result in driving stresses much less than 90 percent (46%-58%) of the specified yield stress. If other H-Piles are utilized that are not shown in the table, driving stresses should be held to approximately this same range.

11.3.1.18 Construction Considerations

Construction considerations generally include selection of pile hammers, use of driving formulas and installation of test piles, when appropriate, as described below.

11.3.1.18.1 Pile Hammers

Pile driving hammers are generally powered by compressed air, steam pressure or diesel units. The diesel hammer, a self-contained unit, is the most popular due to its compactness and adoption in most construction codes. Also, the need for auxiliary power is eliminated and the operation cost is nominal. Vibratory and sonic type hammers are employed in special cases where speed of installation is important and/or noise from impact is prohibited. The vibrating hammers convert instantly from a pile driver to a pile extractor by merely tensioning the lift line.

Pile hammers are raised and allowed to fall either by gravity or with the assistance of power. If the fall is due to gravity alone, the hammer is referred to as single-acting. The single-acting hammer is suitable for all types of soil but is most effective in penetrating heavy clays. The major disadvantage is the slow rate of driving due to the relatively slow rate of blows from 50 to 70 per minute. Wisconsin construction specifications call for a minimum hammer weight depending on the required final bearing value of the pile being driven. In order to avoid damage to the pile, the fall of the gravity hammer is limited to 10 feet.

If power is added to the downward falling hammer, the hammer is referred to as double-acting. This type of hammer works best in sandy soil but also performs well in clay. Double-acting hammers deliver 100 to 250 blows per minute, which increases the rate of driving considerably over the single-acting hammers. Wisconsin construction specifications call for a rated minimum energy of 15 percent of the required bearing of the pile. A rapid succession of blows at a high velocity can be extremely inefficient, as the hammer bounces on heavy piles.

Differential-acting hammers overcome the deficiencies found with both single- and double-acting hammers by incorporating higher frequency of blows and more efficient transfer of energy. The steam cycle, which is different from that of any other hammer, makes the lifting area under the piston independent of the downward thrusting area above the piston. Sufficient force can be applied for lifting and accelerating these parts without affecting the dead weight needed to resist the reaction of the downward acceleration force. The maximum delivered energy per blow is the total weight of the hammer plus the weight of the downward steam force times the length of the stroke.

The contractor’s selection of the pile hammer is generally dependent on the following:

- The hammer weight and rated energy are selected on the basis of supplying the maximum driving force without damaging the piles.
- The hammer types dictated by the construction specification for the given pile type.
- The hammer types available to the contractor.
- Special situations, such as sites adjacent to existing buildings, that require consideration of vibrations generated from the driving impact or noise levels. In these

instances, reducing the hammer size or choosing a double-acting hammer may be preferred over a single-acting hammer. Impact hammers typically cause less ground vibration than vibratory hammers.

- The subsurface conditions at the site.

- The required final resistance capacity of the pile.

WisDOT specifications require the heads of all piling to be protected by caps during driving. The pile cap serves to protect the pile, as well as modulate the blows from the hammer which helps eliminate large inefficient hammer forces. When penetration-per-blow is used as the driving criteria, constant cap-block material characteristics are required. The cap-block characteristics are also assumed to be constant for all empirical formula computations to determine the rate of penetration equivalent to a particular dynamic resistance.

11.3.1.18.2 Driving Formulas

Formulas used to estimate the bearing capacity of piles are of four general types – empirical, static, dynamic and wave equation.

Empirical formulas are based upon tests under limited conditions and are not suggested for general use.

Static formulas are based on soil stresses and try to equate shaft resistance and point resistance to the load-bearing capacity of the piles.

Dynamic pile driving formulas assume that the kinetic energy imparted by the pile hammer is equal to the nominal pile resistance plus the energy lost during driving, starting with the following relationship:

$$\text{Energy input} = \text{Energy used} + \text{Energy lost}$$

The energy used equals the driving resistance multiplied by the pile movement. Thus, by knowing the energy input and estimating energy losses, driving resistance can be calculated from observed pile movement. Numerous dynamic formulas have been proposed. They range from the simpler Engineering News Record (ENR) Formula to the more complex Hiley Formula. A modified Engineering News Formula was previously used by WisDOT to determine pile resistance capacity during installation. All new designs shall use the modified Gates or WAVE equation for determining the required driving resistance.

The following modified FHWA-Gates Formula is used by WisDOT:

$$R_R = \varphi_{dyn} R_{ndr} = \varphi_{dyn}\left(0.875\left(E_d\right)^{0.5}\log_{10}\left(10/s\right)-50\right)$$

Where:

R_R = Factored pile resistance (tons)

φ_{dyn} = Resistance factor = 0.5 **LRFD [Table 10.5.5.2.3-1]**

R_{ndr} = Nominal pile resistance measured during pile driving (tons)

E_d = Energy delivered by the hammer per blow (lb-foot)

s = Average penetration in inches per blow for the final 10 blows (inches/blow)

Because of the difficulty of evaluating the many energy losses involved with pile driving, these dynamic formulas can only approximate pile driving resistance. These approximate results can be used as a safe means of determining pile length and bearing requirements. Despite the obvious limitations, the dynamic pile formulas take into account the best information available and have considerable utility to the engineer in securing reasonably safe and uniform results over the entire project.

The wave equation can be used to set driving criteria to achieve a specified pile bearing capacity (contact the Bureau of Technical Services, Geotechnical Engineering Unit prior to using the wave equation to set the driving criteria). The wave equation is based upon the theory of longitudinal wave transmission. This theory, proposed by Saint Venant a century ago, did not receive widespread use until the advent of computers due to its complexity. The wave equation can predict impact stresses in a pile during driving and estimate static soil resistance at the time of driving by solving a series of simultaneous equations. An advantage of this method is that it can accommodate any pile shape, as well as any distribution of pile shaft resistance and point resistance. The effect of the hammer and cushion block can be included in the computations.

Dynamic monitoring is performed by a Pile Driving Analyzer (PDA). WisDOT uses the PDA to evaluate the driving criteria, which is set by a wave equation analysis, and in an advisory capacity for evaluating if sufficient pile penetration is achieved, if pile damage has occurred or if the driving system is performing satisfactorily.

The PDA provides a method of dynamic pile testing both for pile design and construction control. Testing is accomplished during pile installation by attaching reusable strain transducers and accelerometers directly on the pile. Piles can be tested while being driven or during restrike. The instrumentation mounted on the pile allows the measurement of force and acceleration signals for each hammer blow. This data is transmitted to a small field computer for processing and recording. Calculations made by the computer based upon one-dimensional wave mechanics provide an immediate readout of maximum stresses in the pile, energy transmitted to the pile and a prediction of the nominal axial resistance of the pile for each hammer impact. Monitoring of the force and velocity wave traces with the computer during driving also enables detection of any structural pile damage that may have occurred. Review of selected force and velocity wave traces are also available to provide additional testing documentation. The PDA can be used on all types of driven piles with any impact type of pile-driving hammer.

11.3.1.18.3 Field Testing

Test piles are employed at a project site for two purposes:

- For test driving, to determine the length of pile required prior to placing purchasing orders.

- For load testing, to verify actual pile capacity versus design capacity for nominal axial resistance.

11.3.1.18.3.1 Installation of Test Piles

Test piles are not required for spliceable types of piles. Previous experience indicates that contractors typically order total plan quantities for cast-in-place or steel H-piling in 60-foot lengths. The contractor uses one of the driven structure piles as a test pile at each designated location.

Test piling should be driven near the location of a soil boring where the soil characteristics are known and representative of the most unfavorable conditions at the site. The test pile must be exactly the same type and dimension as the piles to be used in the construction and installed by the same equipment and manner of driving. A penetration record is kept for every 1 foot of penetration for the entire length of pile. This record may be used as a guide for future pile driving on the project. Any subsequent pile encountering a smaller resistance is considered as having a smaller nominal resistance capacity than the test pile.

11.3.1.18.3.2 Static Pile Load Tests

A static pile load test is usually conducted to furnish information to the geotechnical engineer to develop design criteria or to obtain test data to substantiate nominal resistance capacity for piles. A static pile load test is the only reliable method of determining the nominal bearing resistance of a single pile, but it is expensive and can be quite time consuming. The decision to embark on an advance test program is based upon the scope of the project and the complexities of the foundation conditions. Such test programs on projects with large numbers of displacement piling often result in substantial savings in foundation costs, which can more than offset the test program cost. WisDOT has only performed a limited number of pile load tests on similar type projects.

Static pile load testing generally involves the application of a direct axial load to a single vertical pile. However, static pile load testing can involve uplift or axial tension tests, lateral tests applied horizontally, group tests or a combination of these applied to battered piles. Most static test loads are applied with hydraulic jacks reacting against either a stable loaded platform or a test frame anchored to reaction piles.

The basic information to be developed from the static pile load test is usually the deflection of the pile head under the test load. Movement of the head is caused by elastic deformation of the piles and the soil. Soil deformation may cause undue settlement and must be guarded against. The amount of deformation is the significant value to be obtained from load tests, rather than the total downward movement of the pile head. Static pile load tests are typically performed by loading to a given deflection value.

It is impractical to test every pile on a project. Therefore, test results can be applied to other piles or pile groups providing that the following conditions exist:

- The other piles are of the same type, material and size as the test piles.
- Subsoil conditions are comparable to those at the test pile locations.
- Installation methods and equipment used are the same as, or comparable to, those used for the test piles.
- Piles are driven to the same penetration depth or resistance or both as the test piles to compensate for variations in the vertical position and density of the bearing strata.

11.3.1.19 Construction Monitoring for Economic Evaluation of Deep Foundations

The goal of the foundation design is to provide the most efficient and economical design for the subsurface conditions. The design of pile-supported foundations is influenced by the resistance factor, which is generally a function of pile resistance determination during installation. The discussion in 11.3.1.14 presents the definition of resistance factors. From a practical point of view the resistance factor for a deep foundation is the relationship between the Factored Axial Compression Resistance (FACR) and the Required Driving Resistance (RDR). The potential resistance factors (see Table 11.3-1) for use in deep foundation design are as follows:

Methods Used to Determine Required Driving Resistance	Resistance Factor
FHWA-modified Gates dynamic pile driving formula (end of drive condition only).	0.50
Driving criteria established by dynamic test [Pile Driving Analyzer, (PDA)] with signal matching [CAse Pile Wave Analysis Program, (CAPWAP)] at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles LRFD[Table 10.5.5.2.3-1]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.65
Static Pile Load Test(s) and dynamic test (PDA) with signal matching (CAPWAP) at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles LRFD[Table 10.5.5.2.3-1]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.80

Table 11.3-6
Resistance Factors and Deep Foundation Methods of Construction Monitoring

The typical method for a majority of the Department's deep foundation substructures is using the modified Gates dynamic formula to determine the RDR and to use a resistance factor of 0.50. A comparison should be made between the use of the modified Gates and the use of the PDA with CAPWAP or the use of the Static Pile Load Test and the PDA with CAPWAP to determine which method is the most economical.

There are two possible methods available to economically use the PDA with CAPWAP to determine the required driving resistance, which allows the use of a resistance factor of 0.65.

Method 1: Reduce the number of piles in the substructure by driving the piles to the same RDR as using the modified Gates, but then increasing the FACR used in design. This is possible because the department has set a maximum value on the RDR, which when converted to the FACR is less than the structural capacity of the piles. This is true for all H-piles, and for some CIP piles when the FACR is controlled by the maximum allowable compression stress during driving based on 90 percent of the specified yield stress of steel.

Method 2: Drive each pile to a lower RDR, which should result in a shorter pile length. The number of piles per substructure would remain the same. The design estimated pile lengths are a function of the assumed soil conditions and the required driving resistance. The as-built pile lengths are a function of the actual soil conditions encountered and the contractor's hammer selection.

The department recommends Method 1 when evaluating the potential economic benefits of using the PDA with CAPWAP, because of the difficultly in accurately predicting pile lengths.

The method used to compare modified Gates to Static Pile Load Test(s) and the PDA with CAPWAP, which allows the use of a resistance factor of 0.80, would follow the procedures described in Method 1 used in the PDA with CAPWAP, reducing the number of piles per substructure. The number of static load test(s) will be a function of the size and number of substructures, the general spatial extent of the area in question and the variability of the subsurface conditions in the area of interest.

The costs to be included in the economic evaluation include the cost of the piling, the cost for the Department/Consultant to monitor the test piles, the cost for the Consultant CAPWAP evaluation (the Department does not currently have this capability), the unit costs for the contractor's time for driving and redriving the test piles, and the cost for the static pile load test(s).

Once the investigation of the subsurface conditions has been completed the geotechnical engineer and the structure engineer should discuss the potential for cost savings by increasing the resistance factor. The Bureau of Structures, Geotechnical Engineering Unit and the Region should be included in the discussion and should be part of the decision. Generally, the larger the project, the greater the potential for significant savings. The Department has two PDA's; therefore, the project team should contact the Geotechnical Engineering Unit (608-246-7940) to evaluate resources prior to incorporation of an increased resistance factor in the foundation design. PDA monitoring may be completed by Department or consultant personnel.

The following two examples use Method 1 to illustrate the potential cost savings/expenses for PDA with CAPWAP:

Pier
Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot. (**Note:** It is realized that for pier design the number of piles is not exclusively related to the vertical load, but this example is simplified for illustrative purposes).
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 110 tons = 32 piles <u>Pile Cost = 32 piles x 100 feet x $40/ft = $128,000</u> Total Cost = $128,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles Pile Cost = 25 piles x 100 feet x $40/ft = $100,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 <u>CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400</u> Total Cost = $103,000
PDA/CAPWAP Savings = $25,000/pier
Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Abut = 980 tons, Number of Piles = 980 tons / 110 tons = 9 piles Total Cost = 9 piles x 100 feet x $40/ft = $36,000
PDA/CAPWAP:

RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 143 tons = 7 piles, however because of maximum spacing requirements the design will need 8 piles. Pile Cost = 8 piles x 100 feet x $40/ft = $32,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400 Total Cost = $35,000
PDA/CAPWAP Cost = $1000/abutment **Note:** For a three span bridge, with 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $52,000. For a two span bridge, with 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $5,400. Bid prices based on 2014-2015 cost data.

Table 11.3-7

Economical Evaluation for Deep Foundations with Two Construction Monitoring Methods

11.3.2 Drilled Shafts

11.3.2.1 General

Drilled shafts are generally large diameter, cast-in-place, open ended, cased concrete piles which are designed to carry extremely heavy loads. Drilled shafts can be the most economical foundation alternative at sites where foundation loads are carried to bearing on dense strata or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or expensive to construct, and where high overturning moments must be resisted.

Drilled shafts are installed by removing soil and rock using drilling methods or other excavation techniques and constructing the foundation element in the excavated hole. The excavated hole may be supported using temporary or permanent casing, drilling slurry or other methods. The hole is then filled with a reinforcement cage and cast-in-place concrete. Drilled shafts are non-displacement elements since the soil volume required for the element is physically removed prior to installation. Thus the effective normal stress adjacent to the pile remains unchanged or is reduced (due to expansion of the soil into the hole before insertion/construction of the load bearing element), and the soil properties and pore water pressure adjacent to the foundation elements are not significantly impacted.

Because drilled shafts do not require a hammer for installation and do not displace the soil, they typically have much less impact on adjacent structures. Depending on the excavation technique used, they can penetrate significant obstructions. Because the method of construction often allows a decrease in the effective stress immediately adjacent to and beneath the tip of the foundation element, the resistance developed will often be less than an equivalently sized driven pile.

Drilled shafts are generally considered fixed to the substructure unit if the reinforcing steel from the shaft is fully developed within the substructure unit.

Drilled shafts vary in diameter from approximately 2.5 to 10 feet. Drilled shafts with diameters greater than 6 feet are generally referred to as piers. Shafts may be designed to transfer load to the bearing stratum through side friction, point-bearing or a combination of both. The drilled shaft may be cased or uncased, depending on the subsurface conditions and depth of bearing.

Drilled shafts have been used on only a small number of structures in Wisconsin. For unusual site conditions, the use of drilled shafts may be advantageous. Design methodologies for drilled shafts can be found in LRFD 10.8 Drilled Shafts and *Drilled Shafts: Construction Procedures and LRFD Design Methods*. FHWA Publication NHI-10-016, FHWA GEC 010. 2010.

Strength limit states for drilled shafts are evaluated in the same way as for driven piles. Drivability is not required to be evaluated. The structural resistance of drilled shafts is evaluated in accordance with **LRFD [5.7 and 5.8]**. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or tension.

11.3.2.2 Resistance Factors

Resistance factors for drilled shafts are presented in Table 11.3-8 and are selected based on the method used to determine the nominal (ultimate) resistance capacity of the drilled shaft. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal shaft resistance. As with driven piles, the selection of a geotechnical resistance factor should be based on the intended method of resistance verification in the field. Because of the cost and difficulty associated with testing drilled shafts, much more reliance is placed on static analysis methods.

Condition/Resistance Determination Method				Resistance Factor
Static Analysis – Used in Design Phase	Nominal Resistance of Single-Drilled Shaft in Axial Compression, φ_{stat}	Shaft Resistance in Clay	Alpha Method	0.45
		Point Resistance in Clay	Total Stress	0.40
		Shaft Resistance in Sand	Beta Method	0.55
		Point Resistance in Sand	O'Neill and Reese	0.50
		Shaft Resistance in IGMs	O'Neill and Reese	0.60
		Point Resistance in IGMs	O'Neill and Reese	0.55
		Shaft Resistance in Rock	Horvath and Kenney O'Neill and Reese	0.55
			Carter and Kulhawy	0.50
		Point Resistance in Rock	Canadian Geotech. Soc. Pressuremeter Method O'Neill and Reese	0.50
	Block Failure, φ_{bl}	Clay		0.55
	Uplift Resistance of Single-Drilled Shaft, φ_{up}	Clay	Alpha Method	0.35
		Sand	Beta Method	0.45
		Rock	Horvath and Kenney Carter and Kulhawy	0.40
	Group Uplift Resistance, φ_{ug}	Sand and Clay		0.45
	Horizontal Geotechnical Resistance of Single Shaft or Pile Group	All Soil Types and Rock		1.0

Table 11.3-8

Geotechnical Resistance Factors for Drilled Shafts **LRFD [Table 10.5.5.2.4-1]**

For drilled shafts, the base geotechnical resistance factors in Table 11.3-8 assume groups containing two to four shafts, which are slightly redundant. For groups containing at least five elements, the base geotechnical resistance factors in Table 11.3-8 should be increased by 20%.

WisDOT policy item:

When a bent contains at least 5 columns (where each column is supported on a single drilled shaft) the resistance factors in Table 11.3-8 should be increased up to 20 percent for the Strength Limit State.

For piers supported on a single drilled shaft, the resistance factors in Table 11.3-8 should be decreased by 20 percent for the Strength Limit State. Use of single drilled shaft piers requires approval from the Bureau of Structures.

Resistance factors for structural design of drilled shafts are obtained from **LRFD [5.5.4.2.1]**.

11.3.2.3 Bearing Resistance

Most drilled shafts provide geotechnical resistance in both end bearing and side friction. Because the rate at which side friction mobilizes is usually much higher than the rate at which end bearing mobilizes, past design practice has been to ignore either end bearing for shafts with significant sockets into the bearing stratum or to ignore skin friction for shafts that do not penetrate significantly into the bearing stratum. This makes evaluation of the geotechnical resistance slightly more complex, because in most cases it is not suitable to simply add the nominal (ultimate) end bearing resistance and the nominal side friction resistance in order to obtain the nominal axial geotechnical resistance.

When computing the nominal geotechnical resistance, consideration must be given to the anticipated construction technique and the level of construction control. If it is anticipated to be difficult to adequately clean out the bottom of the shafts due to the construction technique or subsurface conditions, the end bearing resistance may not be mobilized until very large deflections have occurred. Similarly, if construction techniques or subsurface conditions result in shaft walls that are very smooth or smeared with drill cuttings, side friction may be far less than anticipated.

Because these resistances mobilize at different rates, it may be more appropriate to add the ultimate end bearing to that portion of the side resistance remaining at the end of bearing failure. Or it may be more appropriate to add the ultimate side resistance to that portion of the end bearing mobilized at side resistance failure. Note that consideration of deflection, which is a service limit state, may control over the axial geotechnical resistance since displacements required to mobilize the ultimate end bearing can be excessive. Shaft Resistance

The shaft resistance is estimated by summing the friction developed in each stratum. When drilled shafts are socketed in rock, the shaft resistance that is developed in soil is generally ignored to satisfy strain compatibility. The following analysis methods are typically used to compute the static shaft resistance in soil and rock:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**
- Horvath and Kenny method for rock, as specified in **LRFD [10.8.3.5.4]**

11.3.2.3.1 Point Resistance

The following analysis methods are typically used to compute the static shaft resistance in soil:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**

The ultimate unit point resistance of a drilled shaft in intact or tightly jointed rock is computed as 2.5 times the unconfined compressive strength of the rock. For rock containing open or filled joints, the geomechanics RMR system is used to characterize the rock, and the ultimate point resistance in rock can be computed as specified in **LRFD [10.8.3.5.4c]**.

11.3.2.3.2 Group Capacity

For drilled shaft groups bearing in cohesive soils or ending in a strong layer overlying a weaker layer, the axial resistance is determined using the same approach as used for driven piles. For drilled shaft groups in cohesionless soil, a group efficiency factor is applied to the ultimate resistance of a single drilled shaft. The group efficiency factor is a function of the center-to-center shaft spacing and is linearly interpolated between a value of 0.65 at a center-to-center spacing of 2.5 shaft diameters and a value of 1.0 at a center-to-center spacing of 6.0 shaft diameters. This reduction is more than for driven piles at similar spacing, because construction of drilled shafts tends to loosen the soil between the shafts rather than densify it as with driven piles.

11.3.2.4 Lateral Load Resistance

Because drilled shafts are made of reinforced concrete, the lateral analysis should consider the nonlinear variation of bending stiffness with respect to applied bending moment. At small applied moments, the reinforced concrete section performs elastically based on the size of the section and the modulus of elasticity of the concrete. At larger moments, the concrete cracks in tension and the stiffness drops significantly.

11.3.2.5 Other Considerations

Detailing of the reinforcing steel in a drilled shaft must consider the constructability of the shaft. The reinforcing cages must be stiff enough to resist bending during handling and concrete placement. In addition, the spaces between reinforcement bars must be kept large enough to permit easy flow of the concrete from the center of shaft to the outside of shaft. These two requirements will generally force the use of larger, more widely spaced longitudinal and transverse reinforcement bars than would be used in the design of an above-grade column. In addition, when using hooked bars to tie the shaft to the foundation, consideration must also be given to concrete placement requirements and temporary casing removal requirements.

11.3.3 Micropiles

11.3.3.1 General

In areas of restricted access, close proximity to settlement sensitive existing structures or difficult geology, micropiles may be considered when determining the recommended foundation type. Although typically more expensive than driven pile, constructability considerations may warrant selection of micropiles as the preferred foundation type. A micropile is constructed by drilling a borehole with drill casing, placing reinforcement and grouting the hole. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can be installed in areas with restricted access and vertical clearance. Drill casing permits installation in poor ground conditions. Micropiles are installed with the same type of equipment that is used for ground anchor and grouting projects. Micropiles can be either vertical or battered.

Micropiles are used for structural support of new structures, underpinning existing structures, scour protection and seismic retrofit at existing structures. Micropiles are also used to create a reinforced soil mass for ground stabilization.

With a micropile's smaller cross-sectional area, the pile design is more frequently governed by structural and stiffness considerations. Due to the small pile diameter, point resistance is usually disregarded for design. Steel casing for micropiles is commonly delivered in 5 to 20 foot long flush-joint threaded sections. The casing is typically 5.5 to 12 inches in diameter, with yield strength of 80 ksi. Grout is mixed neat with a water/cement ratio on the order of 0.45 and an unconfined compressive strength of 4 to 6 ksi. Grade 60, 90 and 150 single reinforcement bars are generally used with centralizers.

Grout/ground bond capacity varies directly with the method of placement and pressure used to place the grout. Common methods include grout placement under gravity head, grout placement under low pressure as temporary drill steel is removed and grout placement under high pressure using a packer and regrout tube. Some regrout tubes are equipped to allow regrouting multiple times to increase pile capacity.

11.3.3.2 Design Guidance

Micropiles shall be designed in conformance with the current *AASHTO LRFD* and in accordance with the WisDOT Bridge Manual. Design guidelines for micropiles are provided in FHWA Publication No. FHWA-NHI-05-039.

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

Augered cast-in-place (ACIP) piles are installed by drilling a hole with a hollow stem auger. When the auger reaches a design depth (elevation) or given torque, sand-cement grout or concrete is pumped through the hollow-stem auger while the auger is withdrawn from the ground. Reinforcement steel can be placed while the grout is still fluid. A single reinforcement bar can also be installed inside the hollow stem auger before the auger is extracted. ACIP piles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can also be installed in areas with restricted access and vertical clearance. Temporary casing is not required. In many situations, these foundation systems can be constructed more quickly and less expensively than other deep foundation alternatives.

ACIP piles are generally available in 12- to 36-inch diameters and typically extend to depths of 60 to 70 feet. In some cases, ACIP piles have been installed to depths of more than 100 feet. The torque capacity of the drilling equipment may limit the available penetration depth of ACIP piles, especially in stiff to hard cohesive soil. Typical Wisconsin bridge contractors do not own the necessary equipment to install this type of pile.

ACIP piles may be more economical; however, there is a greater inherent risk in their installation from the quality control standpoint. There is currently no method available to determine pile capacity during construction of ACIP piles. WisDOT does not generally use this pile type unless there are very unusual design/site requirements.

11.3.4.2 Design Guidance

In the future, the FHWA will distribute a Geotechnical Engineering Circular that will provide design and construction guidance for ACIP piles. WisDOT plans to reassess the use of ACIP piles at that time.

11.4 References

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2. American Association of State Highway and Transportation Officials. *Standard Specifications for Highway Bridges*.

3. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications*.

4. Armour, T., P. Groneck, J. Keeley and S. Sharma. *Micropile Design and Construction Guidelines, Implementation Manual*. FHWA Publication SA-97-070. June 2000.

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8. Lagrasse, P.F., J.D. Schall and E.V. Richardson. *Hydraulic Engineering Circular No. 20: Stream Stability at Highway Structures*, 3rd Edition. FHWA Publication NHI 01-002. March 2001.

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10. Leonards, G.A. *Foundation Engineering*. McGraw Hill. 1962.

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12. *Micropile Design and Construction Reference Manual*. FHWA Publication NHI-05-039. 2005.

13. O'Neil, M.W. and L.C. Reese. *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication IF-99-025. August 1999.

14. O'Neil, M.W., F.C. Townsend, K.M. Hassan, A. Butler and P.S. Chan. *Load Transfer for Drilled Shafts in Intermediate Geomaterials*. FHWA Publication RD-95-172. November 1996.

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16. Sabatini, P.J. et al. *Geotechnical Engineering Circular No. 5: Evaluation of Soil and Rock Properties*. FHWA Publication IF-02-034. April 2002.

17. Terzaghi, K. and R.B. Peck. *Soil Mechanics in Engineering Practice*, 2nd Edition. Wiley. 1967.

18. Tschebotrarioff, G.P. *Foundations, Retaining and Earth Structures*, 2nd Edition. McGraw Hill. 1973.

19. *Drilled Shafts: Construction Procedures and LRFD Design Methods*. FHWA Publication NHI-10-016, FHWA GEC 010. 2010.

11.5 Design Examples

WisDOT will provide design examples.

This section will be expanded later when the design examples are available.

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Table of Contents

12.1 General

Abutments are used at the ends of bridges to retain the embankment and to carry the vertical and horizontal loads from the superstructure to the foundation, as illustrated in Figure 12.1-1. The design requirements for abutments are similar to those for retaining walls and for piers; each must be stable against overturning and sliding. Abutment foundations must also be designed to prevent differential settlement and excessive lateral movements.

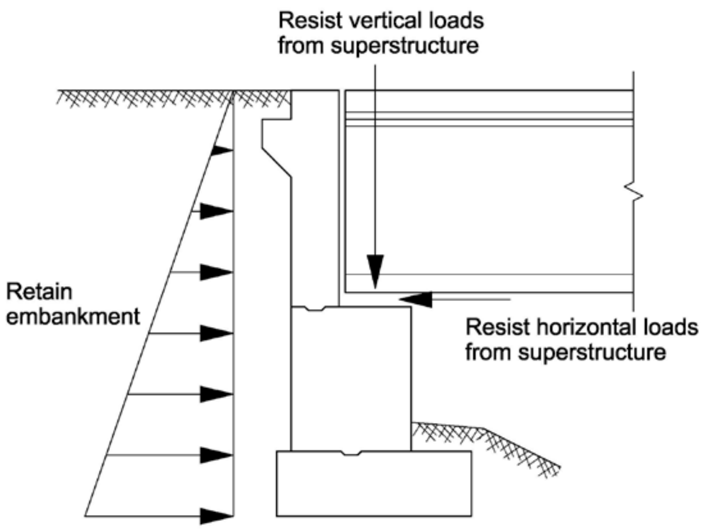


Figure 12.1-1
Primary Functions of an Abutment

The components of a typical abutment are illustrated in Figure 12.1-2.

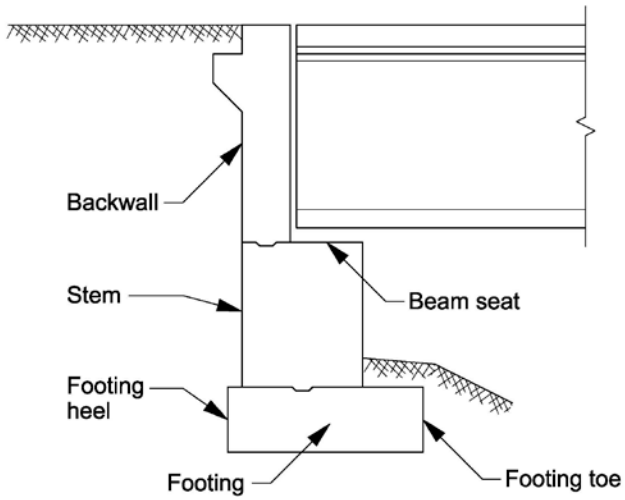


Figure 12.1-2
Components of an Abutment

Many types of abutments can be satisfactorily utilized for a particular bridge site. Economics is usually the primary factor in selecting the type of abutment to be used. For river or stream crossings, the minimum required channel area and section are considered. For highway overpasses, minimum horizontal clearances and sight-distances must be maintained.

An abutment built on a slope or on top of a slope is less likely to become a collision obstacle than one on the bottom of the slope and is more desirable from a safety standpoint. Aesthetics is also a factor when selecting the most suitable abutment type.

12.2 Abutment Types

Several different abutment types can be used, including full-retaining, semi-retaining, sill, spill-through or open, pile-encased and special designs. Each of these abutment types is described in the following sections.

12.2.1 Full-Retaining

A full-retaining abutment is built at the bottom of the embankment and must retain the entire roadway embankment, as shown in Figure 12.2-1. This abutment type is generally the most costly. However, by reducing the span length and superstructure cost, the total structure cost may be reduced in some cases. Full-retaining abutments may be desirable where right of way is critical.

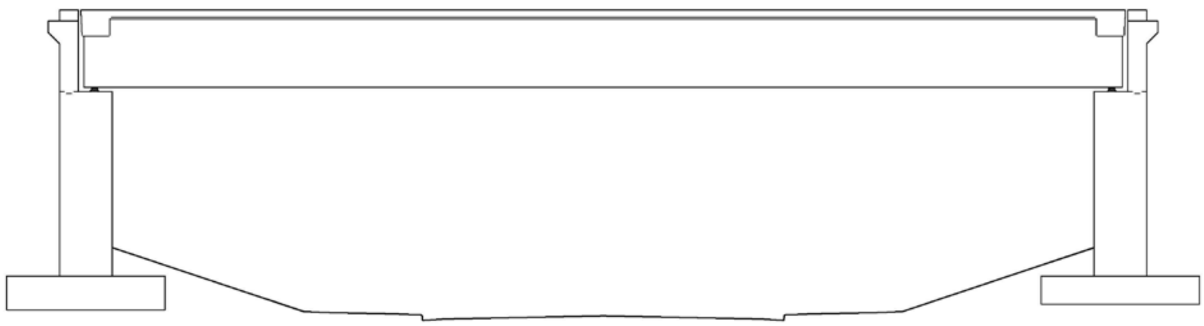


Figure 12.2-1
Full-Retaining Abutment

Rigid-frame structures use a full-retaining abutment poured monolithically with the superstructure. If both abutments are connected by fixed bearings to the superstructure (as in rigid frames), the abutment wings are joined to the body by a mortised expansion joint. For a non-skewed abutment, this enables the body to rotate about its base and allows for superstructure contraction and expansion due to temperature and shrinkage, assuming that rotation is possible.

An objectionable feature of full-retaining abutments is the difficulty associated with placing and compacting material against the body and between the wing walls. It is possible that full-retaining abutments may be pushed out of vertical alignment if heavy equipment is permitted to work near the walls, and this temporary condition is not accounted for in a temporary load combination. The placement of the embankment after abutment construction may cause foundation settlement. For these reasons, as much of the roadway embankment as practical should be in place before starting abutment construction. Backfilling above the beam seat is prohibited until the superstructure is in place.

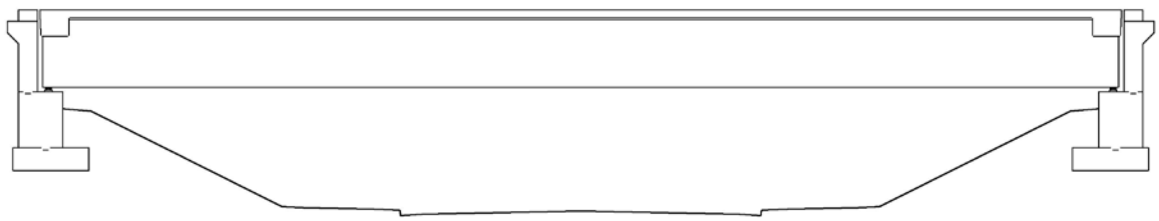
Other disadvantages of full-retaining abutments are:

- Minimum horizontal clearance

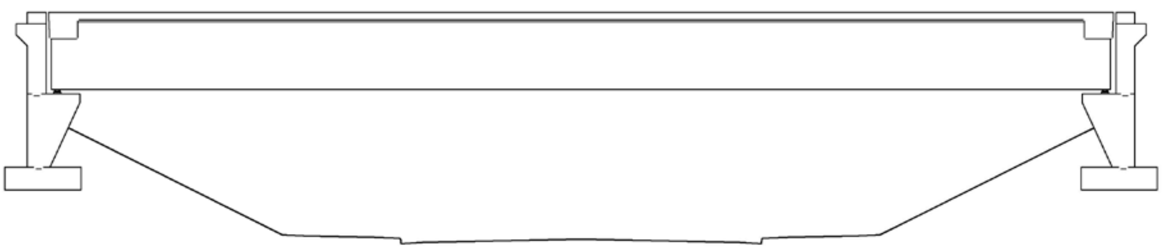
- Minimum sight distance when roadway underneath is on a curved alignment
- Collision hazard when abutment front face is not protected
- Settlement

12.2.2 Semi-Retaining

The semi-retaining abutment (Types A3 and A4) is built somewhere between the bottom and top of the roadway embankment, as illustrated in Figure 12.2-2. It provides more horizontal clearance and sight distance than a full-retaining abutment. Located on the embankment slope, it becomes less of a collision hazard for a vehicle that is out of control.



A3



A4

Figure 12.2-2
Semi-Retaining Abutment

The description of full-retaining abutments in 12.2.1 generally applies to semi-retaining abutments as well. They are used primarily in highway-highway crossings as a substitute for a shoulder pier and sill abutment. Semi-retaining abutments generally are designed with a fixed base, allowing wing walls to be rigidly attached to the abutment body. The wings and the body of the abutment are usually poured monolithically.

For deep girder bridges (girder height > 60 inches) the aesthetic appearance of the A4 abutment is minimized and the A3 abutment should be considered.

12.2.3 Sill

The sill abutment (Type A1) is constructed at the top of the slope after the roadway embankment is close to final grade, as shown in Figure 12.2-3. The sill abutment helps avoid many of the problems that cause rough approach pavements. It eliminates the difficulties of obtaining adequate compaction adjacent to the relatively high walls of closed abutments. Since the approach embankment may settle by forcing up or bulging up the slope in front of the abutment body, a berm is often constructed at the front of the body. The weight of the berm helps prevent such bulging.

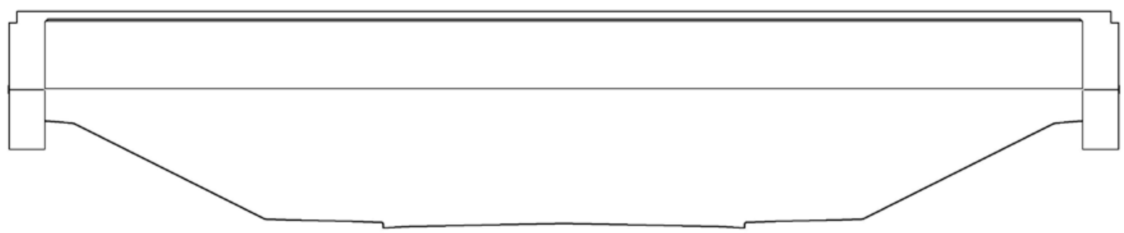


Figure 12.2-3
Sill Abutment

Sill abutments are the least expensive abutment type and are usually the easiest to construct. However, this abutment type results in a higher superstructure cost, so the overall cost of the structure should be evaluated with other alternatives.

For shallow superstructures where wing piles are not required, the Type A1 abutment is used with a fixed seat. This minimizes cracking between the body wall and wings. However, for shallow superstructures where wing piles are required, the Type A1 abutment is used with a semi-expansion seat. This allows superstructure movement, and it reduces potential cracking between the wings and body.

The parallel-to-abutment-centerline wings or elephant-ear wings, as shown on the Standard Details for Wings Parallel to A1 Abutment Centerline, should be used for grade separations when possible. This wing type is preferred because it increases flexibility in the abutment, it simplifies compaction of fill, and it improves stability. Wings parallel to the roadway are still required at stream crossings where high water may be a problem.

12.2.4 Spill-Through or Open

A spill-through or open abutment is mostly used where an additional span may be added to the bridge in the future. It may also be used to satisfy unique construction problems. This abutment type is situated on columns or stems that extend upward from the natural ground. It is essentially a pier being used as an abutment.

It is very difficult to properly compact the embankment materials that must be placed around the columns and under the abutment cap. Early settlement and erosion are problems frequently encountered with spill-through or open abutments.

If the abutment is to be used as a future pier, it is important that the wings and backwall be designed and detailed for easy removal. Construction joints should be separated by felt or other acceptable material. Reinforcing steel should not extend through the joints. Bolts with threaded inserts should be used to carry tension stresses across joints.

12.2.5 Pile-Encased

Pile-encased abutments (Type A5) should only be used where documented cost data shows them to be more economical than sill abutments due to site conditions. For local roads right-of-way acquisition can be difficult, making the A5 a good option. Requiring crane access from only one side of a stream may be another reason to use a single span bridge with A5 abutments, as would savings in railing costs. Steeper topography may make A5 abutments a more reasonable choice than sill abutments. In general, however, using sill abutments with longer bridges under most conditions has cost advantages over using the Type A5 abutments. Type A5 abutments may require additional erosion control measures that increase construction cost.

The wall height of pile-encased abutments is limited to a maximum of 10 feet since increased wall height will increase soil pressure, resulting in uneconomical pile design due to size or spacing requirements. Reinforcement in the abutment body is designed based on live load surcharge and soil pressure on the back wall.

Pile-encased abutments are limited to a maximum skew of 15 and 30 degrees with girder structures and slab structures, respectively, in order to limit damage due to thermal expansion and contraction of the superstructure. Wing skew angles are at 45 degrees relative to the body to prevent cracking between the abutment body and wings.

12.2.6 Special Designs

In addition to the standard abutment types described in the previous sections, many different styles and variations of those abutment types can also be designed. Such special abutment designs may be required due to special aesthetic requirements, unique soil conditions or unique structural reasons. Special designs of abutments require prior approval by the Bureau of Structures Development Chief.

12.3 Types of Abutment Support

Piles, drilled shafts and spread footings are the general types of abutment support used. This section provides a brief description of each type of abutment support.

WisDOT policy item:

Geotechnical design of abutment supports shall be in accordance with the 4th Edition of the AASHTO LRFD Bridge Design Specifications for Highway Bridges. No additional guidance is available at this time.

Structural design of abutment supports shall be in accordance with LRFD, as specified in the 4th Edition of AASHTO LRFD Bridge Design Specifications

12.3.1 Piles or Drilled Shafts

Most abutments are supported on piles to prevent abutment settlement. Bridge approach embankments are usually constructed of fill material that can experience settlement over several years. This settlement may be the result of the type of embankment material or the original foundation material under the embankment. By driving piles through the embankment and into the original ground, abutments usually do not settle with the embankment. A settling embankment may be resisted by the abutment piles through friction between the piles and fill material. The added load to friction piles and the need for preboring should be considered.

It is generally not necessary to prebore non-displacement piles for any fill depths, and it is not necessary to prebore displacement piles for fill depths less than 15 feet below the bottom of footing. However, for some problem soils this may not apply. See the soils report to determine if preboring is required. If required, the Special Provisions must be written with preboring guidelines.

Battered piles may cause more of a problem than vertical piles and are given special consideration. When driving battered piles, reduced hammer efficiency may be experienced, and battered piles should not be considered when negative skin friction loads are anticipated.

Fill embankments frequently shift laterally, as well as vertically. A complete foundation site investigation and information on fill material is a prerequisite for successful pile design.

Piles placed in prebored holes cored into rock do not require driving. The full design loading can be used if the hole is of adequate size to prevent pile hangups and to allow filling with concrete.

Piles in abutments are subject to lateral loads. The lateral resistance on a pile is usually determined from an acceptable level of lateral displacement and not the ultimate load that causes a stress failure in the pile. The lateral resistance on a pile may be more dependent on the material into which the pile is driven than on the pile type. See Chapter 11 – Foundation Support for a more thorough discussion of piles and allowable pile loads.

12.3.2 Spread Footings

Abutments on spread footings are generally used only in cut sections where the original soil can sustain reasonable pressures without excessive settlement. The bearing resistance is determined by the Geotechnical Section or the geotechnical consultant.

With improved procedures and better control of embankment construction, spread footings can be used successfully on fill material. It is important that construction be timed to permit the foundation material to consolidate before the spread footings are constructed. An advantage of spread footings is that the differential settlement between approach fills and abutments is minimized.

The use of spread footings is given greater consideration for simple-span bridges than for continuous-span bridges. However, under special conditions, continuous-span bridges can be designed for small amounts of settlement. Drainage for abutments on spread footings can be very critical. For these reasons, pile footings are usually preferred.

Lateral forces on abutments are resisted by passive earth pressure and friction between the soil and concrete. A shear key provides additional area on which passive earth pressure can act. A berm in front of the abutment may be necessary to prevent a shear failure in the soil along the slope.

12.4 Abutment Wing Walls

This section provides general equations used to compute wing wall lengths, as well as a brief description of wing wall loads and parapets.

12.4.1 Wing Wall Length

Wing walls must be long enough to retain the roadway embankment based on the required roadway slopes. They usually extend parallel to the centerline of the roadway. A slope of 2:1 is usually used, and a slope greater than 2:1 is usually not permitted. Current practices are to round to the nearest 2 feet for wings less than or equal to 12 feet and round to the nearest 4 feet for wings greater than 12 feet. When setting wing wall lengths, be sure that the theoretical slope of the earth does not fall above the bridge seat elevation at the corner.

12.4.1.1 Wings Parallel to Roadway

The calculation of wing wall lengths for wings that are parallel to the roadway is illustrated in Figure 12.4-1 and Figure 12.4-2.

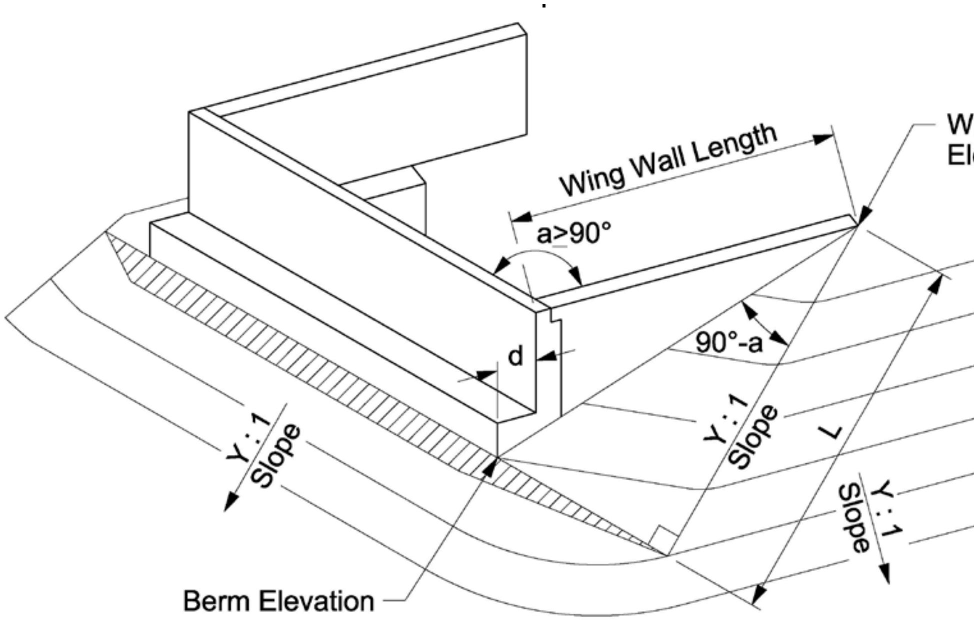


Figure 12.4-1
Wings Parallel to Roadway and Wing Wall Angle ≥ 90°

For wing wall angle, a ≥ 90°:

$$L_{\text{Horizontal Component}} = (\text{Wing Tip Elevation} - \text{Berm Elevation})(Y)$$

$$\text{Wing Wall Length} = \frac{L}{\cos(a - 90^\circ)} - d$$

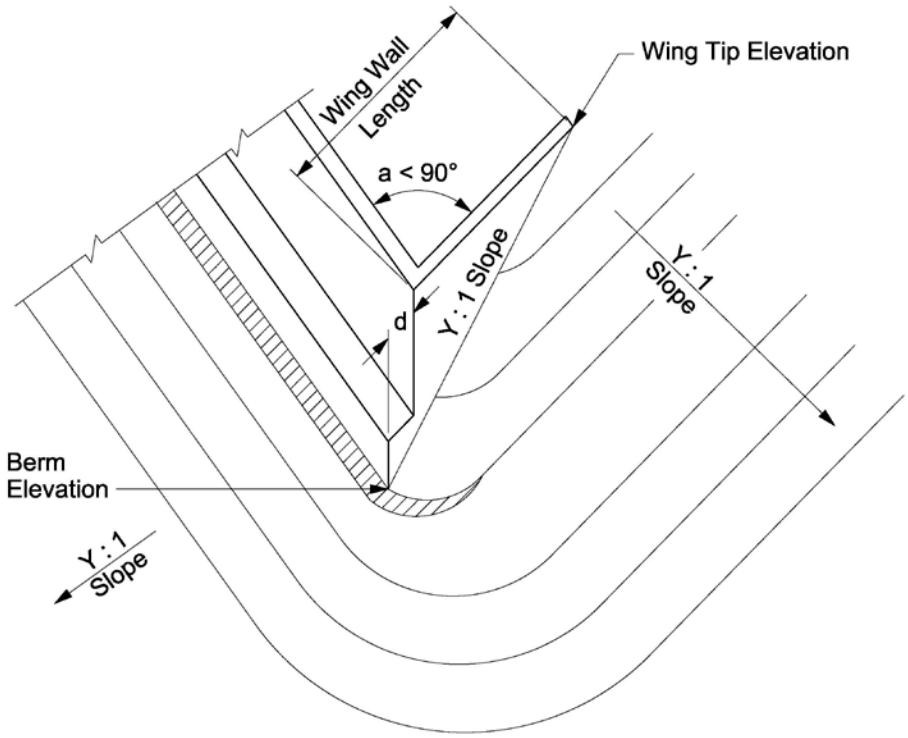


Figure 12.4-2
Wings Parallel to Roadway and Wing Wall Angle < 90°

For wing wall angle, a < 90°:

$$\text{Wing Wall Length} = [(\text{Wing Tip Elevation} - \text{Berm Elevation})(Y)] - d$$

12.4.1.2 Wings Not Parallel to Roadway and Equal Slopes

The calculation of wing wall lengths for wings that are not parallel to the roadway and that have equal slopes is illustrated in Figure 12.4-3.

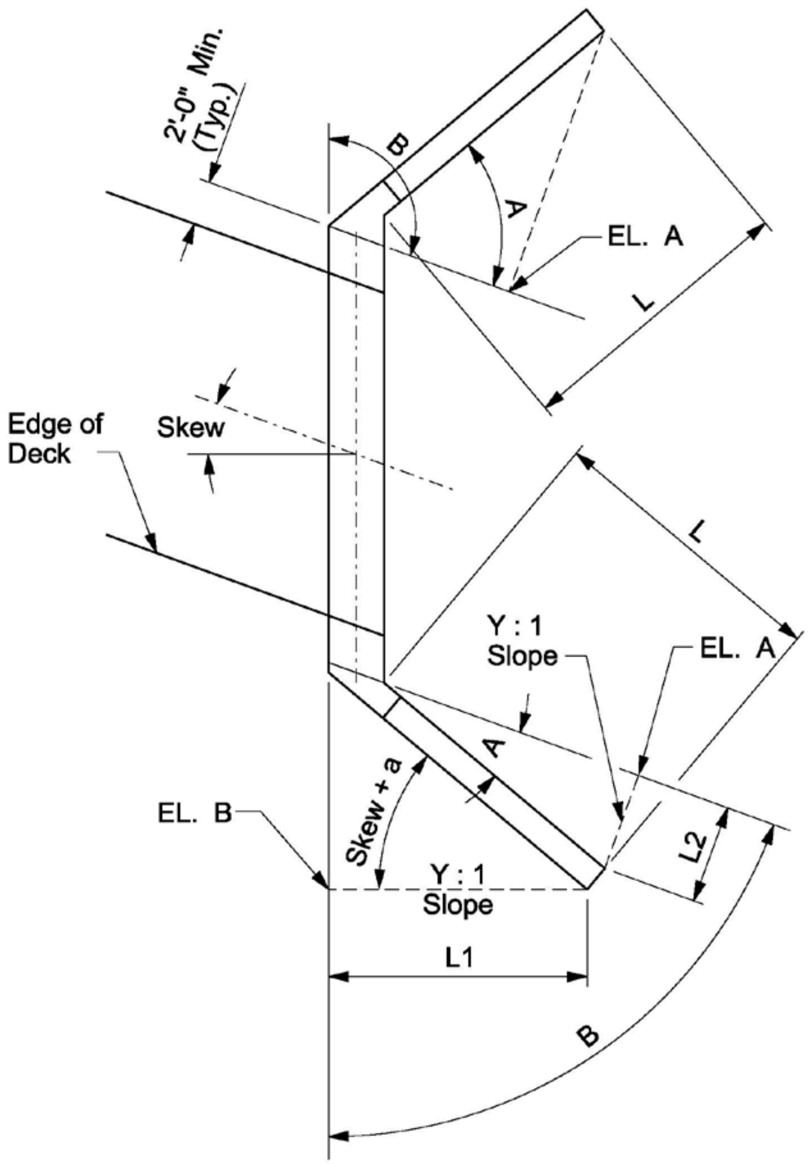


Figure 12.4-3
Wings Not Parallel to Roadway and Equal Slopes

For angle B ≥ 90°:

$$L1 + L2 = (EL.\ A - EL.\ B)(Y)$$

$$\cos(a - Skew) = \frac{L1}{L}$$

$$\sin(a) = \frac{L2}{L}$$

$$L = \frac{Y\,(EL.\ A - EL.\ B)}{\cos(a - Skew) + \sin(a)}$$

For angle B < 90°:

$$L1 + L2 = (EL.\ A - EL.\ B)(Y)$$

$$\cos(Skew + a) = \frac{L1}{L}$$

$$\sin(a) = \frac{L2}{L}$$

$$L = \frac{Y\,(EL.\ A - EL.\ B)}{\cos(Skew + a) + \sin(a)}$$

12.4.2 Wing Wall Loads

Wing walls are designed as retaining walls. Earth loads and surcharge loads are applied to wing walls similar to how they are applied to the stem of a retaining wall. Wing walls are analyzed as cantilevers extending from the abutment body.

The parapet on top of the wing is designed to resist railing loading, but it is not necessary that the railing loads be applied to the wing walls. Railing loads are dynamic or impact loads and are absorbed by the mass of the wing wall and if necessary by passive earth pressure.

The forces produced by the active earth pressure are resisted by the wing piles and the abutment body. Passive earth pressure resistance generally is not utilized, because there is a possibility that the approach fill slopes may slide away from the wings. This may seem like a conservative assumption, but it is justified due to the highly unpredictable forces experienced by a wing wall.

Wing walls without special footings that are poured monolithically with the abutment body are subjected to a bending moment, shear force and torsion. The primary force is the bending moment. Torsion is usually neglected.

The bending moment induced in the cantilevered wing wall by active earth pressure is reduced by the expected lateral resistance of the wing pile group times the distance to the section being investigated. This lateral pile resistance is increased by using battered piles. Individual piles offer little lateral resistance because of small wing deflections. See Chapter 11 – Foundation Support for lateral pile resistance.

12.4.3 Wing Wall Parapets

Steel plate beam guard is used at bridge approaches and is attached to the wing wall parapets. This helps to prevent vehicles from colliding directly into the end of the parapet.

A vehicle striking a guard rail may produce a high-tension force in the guard rail. It is important that sufficient longitudinal parapet steel be provided to resist this force. If the concrete in the parapet is demolished, the longitudinal steel continues to act as a cable guard rail if it remains attached to the steel plate beam guard.

12.5 Abutment Depths, Excavation and Construction

This section describes some additional design considerations for abutments, including depth, excavation and construction.

Abutment construction must satisfy the requirements for construction joints and beam seats presented in 12.9.1 and 12.9.2, respectively.

The abutment body is generally located above the normal water. Refer to the *Standard Specifications* or Special Provisions if part of the abutment body is below normal water.

12.5.1 Abutment Depths

The required depth of the abutment footing to prevent frost damage depends on the amount of water in the foundation material. Frost damage works in two directions. First, ice lenses form in the soil, heaving it upward. These lenses grow by absorbing additional water from below the frost line. Silts are susceptible to heaves, but well-drained sands and dense clays generally do not heave. Second, the direction of frost action is downward. The ice lenses thaw from the top down, causing a layer of water to be trapped near the surface. This water emulsifies the soil, permitting it to flow out from under the footing.

Sill and semi-retaining abutments are constructed on slopes which remain relatively moisture free. Sill abutments have been constructed in all parts of Wisconsin with footings only 2.5 feet below ground and have experienced no frost heave problems.

Full-retaining abutments are constructed at the bottom of embankment slopes, and their footings are more likely to be within a soil of high moisture content. Therefore, footings for full-retaining abutments must be located below the level of maximum frost penetration. Maximum frost penetration varies from 4 feet in the southeastern part of Wisconsin to 6 feet in the northwestern corner.

12.5.2 Abutment Excavation

Abutment excavation is referred to as "Excavation for Structures Bridges." It is measured as a unit for each specific bridge and is paid for at the contract lump sum price.

When a new bridge is constructed, a new roadway approaching the bridge is generally also constructed. Since the roadway contractor and bridge contractor are not necessarily the same, the limits of excavation to be performed by each must be specified. The roadway contractor cuts or fills earth to the upper limits of structural excavation as specified on the bridge plans or in the *Standard Specifications for Highway and Structure Construction*. If the bridge contractor does his work before the roadway contractor or if there is no roadway contract, the upper limit of structural excavation is the existing ground line. For sill abutments, the upper limit is specified in the *Standard Specifications* and need not be shown on the abutment plans.

For semi-retaining and full-retaining abutments, the upper limits are shown on the abutment plans. If a cut condition exists, the upper limit is usually the subgrade elevation and the top surface of the embankment slope (bottom of slope protection). Earth above these limits is removed by the roadway contractor. A semi-retaining or full-retaining abutment placed on fill is considered a unique problem by the design engineer, and limits of excavation must be set accordingly. Construction sequence and type of fill material are considered when setting excavation limits. Slopes greater than 1.5 horizontal to 1 vertical are difficult to construct and generally are not specified. It is sometimes advantageous to have the roadway contractor place extra fill that later must be excavated by the bridge contractor, because the overburden aids in compaction and reduces subsequent settlement.

Lateral limits of excavation are not defined in the *Standard Specifications*. The contractor must excavate whatever is necessary within the right-of-way for the placement of the forms.

12.6 Abutment Drainage and Backfill

This section describes abutment design considerations related to drainage and backfill. The abutment drainage and backfill must be designed and detailed properly to prevent undesirable loads from being applied to the abutment.

12.6.1 Abutment Drainage

Abutment drainage is necessary to prevent hydrostatic pressure and frost pressure. Hydrostatic pressure, including both soil and water, can amount to an equivalent fluid unit weight of soil of 85 pcf. Frost action, which can occur in silty backfill, may result in extremely high pressures. On high abutments, these pressures will produce a very large force which could result in structural damage or abutment movement if not accounted for in the design.

To prevent these additional pressures on abutments, it is necessary to drain away whatever water accumulates behind the body and wings. This is accomplished using a pervious granular fill on the inside face of the abutment. Pipe underdrain must be provided to drain the fill located behind the abutment body and wings. For rehabilitation of structures, provide plan details to replace inadequate underdrain systems.

Past experience indicates that sill abutments are not capable of withstanding hydrostatic pressure on their full height without leaking.

Semi-retaining and full-retaining abutments generally will be overstressed or may slide if subject to large hydrostatic or frost pressures unless accounted for in the design. Therefore, “Pipe Underdrain Wrapped 6-inch” is required behind all abutments. This pipe underdrain is used behind the abutment and outside the abutment to drain the water away. It is best to place the pipe underdrain at the bottom of footing elevation as per standards. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain should be placed higher.

Pipe underdrains and weepholes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks.

12.6.2 Abutment Backfill Material

All abutments and wings shall utilize “Backfill Structure” to facilitate drainage. See Standard Detail 9.01 – Structure Backfill Limits and Notes – for typical pay limits and plan notes.

12.7 Selection of Standard Abutment Types

From past experience and investigations, the abutment types presented in Figure 12.7-1 are generally most suitable and economical for the given conditions. Although piles are shown for each abutment type, drilled shafts or spread footings may also be utilized depending on the material conditions at the bridge site. The chart in Figure 12.7-1 provides a recommended guide for abutment type selection.

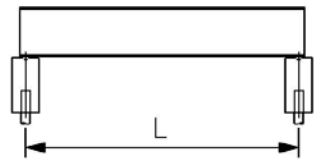
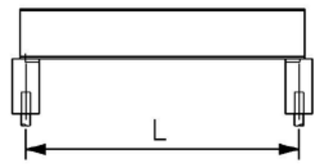
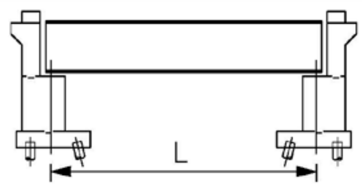
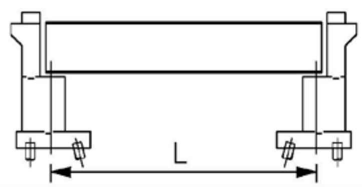
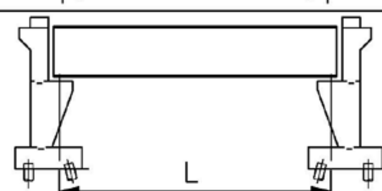
Abutment Arrangements L=Length of continuous superstructure between abutments			Superstructures S=Skew, AL=Abutment Length Concrete Slab Spans	 Prestressed Girders	 Steel Girders
(1) Type A1 with fixed seat		Type A1 with fixed seat	a. L $\le$ 150' S $\le$ 30° AL $\le$ 50'	a. L $\le$ 150' S $\le$ 15° AL $\le$ 50'	a. L $\le$ 150' S $\le$ 15° AL $\le$ 50'
(2) Type A1 with semi-exp. seat		Type A1 with semi-exp. seat	a. L $\le$ 300' S $\le$ 30° AL > 50'	a. L $\le$ 300' S $\le$ 40°	a. L $\le$ 150' S $\le$ 40°
(3) Type A3 with fixed bearing		Type A3 with exp. bearing	Not used	Single span and (S > 40°)	Single span and (L > 150' or S > 40°)
(4) Type A3 with exp. bearing		Type A3 with exp. bearing	b. L >300' and S $\le$ 30° with rigid piers	L>300' or (S > 40° and multi-span)	Multi-span and (L>150' or S > 40°) with rigid piers
(5) Type A4 with exp. bearing		Type A4 with exp. bearing	Not used	c. Based on geometry and economics Girder D < 60"	d. Based on geometry and economics Girder D < 60"
ABUTMENT TYPES					

Figure 12.7-1
Recommended Guide for Abutment Type Selection

Footnotes to Figure 12.7-1:

a. Type A1 fixed abutments are not used when wing piles are required. The semi-expansion seat is used to accommodate superstructure movements and to minimize cracking between the wings and body wall. See Standards for Abutment Type A1 (Integral Abutment) and Abutment Type A1 for additional guidance.

b. Consider the flexibility of the piers when choosing this abutment type. Only one expansion bearing is needed if the structure is capable of expanding easily in one direction. With rigid piers, symmetry is important in order to experience equal expansion movements and to minimize the forces on the substructure units.

c. For two-span prestressed girder bridges, the sill abutment is more economical than a semi-retaining abutment if the maximum girder length is not exceeded. It also is usually more economical if the next girder size is required.

d. For two-span steel structures with long spans, the semi-retaining abutments may be more economical than sill abutments due to the shorter bridge lengths if a deeper girder is required.

12.8 Abutment Design Loads and Other Parameters

This section provides a brief description of the application of abutment design loads, a summary of load modifiers, load factors and other design parameters used for abutment and wing wall design, and a summary of WisDOT abutment design policy items.

12.8.1 Application of Abutment Design Loads

An abutment is subjected to both horizontal and vertical loads from the superstructure. The number and spacing of the superstructure girders determine the number and location of the concentrated reactions that are resisted by the abutment. The abutment also resists loads from the backfill material and any water that may be present.

Although the vertical and horizontal reactions from the superstructure represent concentrated loads, they are commonly assumed to be distributed over the entire length of the abutment wall or stem that support the reactions. That is, the sum of the reactions, either horizontal or vertical, is divided by the length of the wall to obtain a load per unit length to be used in both the stability analysis and the structural design. This procedure is sufficient for most design purposes.

Approach loads are not considered in the example below. However, designers shall include vertical reactions from reinforced concrete approaches as they directly transmit load from the approaches to the abutment. Reinforced concrete approaches include the concrete approach slab system (refer to FDM 14-10-15) and the structural approach slab system (as described in this chapter).

The first step in computing abutment design loads is to compute the dead load reactions for each girder or beam. To illustrate this, consider a 60-foot simple span structure with a roadway width of 44 feet, consisting of steel beams spaced at 9 feet and carrying an HL-93 live loading.

The dead load forces, DC and DW, acting on the abutments shall include reactions from the superstructure. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. If the total DC dead load is 1.10 kips per foot of girder and the total DW dead load is 0.18 kips per foot of girder, then the dead load reaction per girder is computed as follows:

$$R_{DC} = (1.10\ \text{K/ft})\left(\frac{60\ \text{Feet}}{2}\right) = 33.0\ \text{kips}$$

$$R_{DW} = (0.18\ \text{K/ft})\left(\frac{60\ \text{Feet}}{2}\right) = 5.4\ \text{kips}$$

These dead loads are illustrated in Figure 12.8-1. The dead loads are equally distributed over the full length of the abutment.

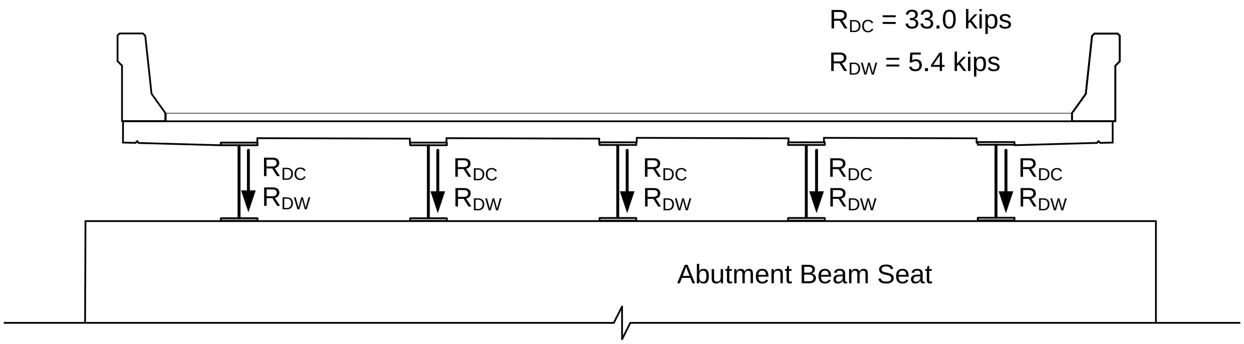


Figure 12.8-1
Dead Load on Abutment Beam Seat

The next step is to compute the live load applied to the abutment. To compute live load reactions to bearings, live load distribution factors must be used to compute the maximum live load reaction experienced by each individual girder. However, to compute live loading on abutments, the maximum number of design lanes are applied to the abutment to obtain the live load per foot of length along the abutment. Live load distribution factors are not used for abutment design, because it is too conservative to apply the maximum live load reaction for each individual girder; each individual girder will generally not experience its maximum live load reaction simultaneously because each one is based on a different configuration of design lane locations.

To illustrate the computation of live loads for abutment design, consider the same 60-foot simple span bridge described previously. Since the roadway width is 44 feet, the maximum number of design lanes is three (44 / 12 = 3.67 ≈ 3 lanes). The backwall live load is computed by placing the three design truck axles along the abutment and calculating the load on a per foot basis. The dynamic load allowance and multiple presence factor shall be included. The load is applied to the entire length of the abutment backwall and is assumed to act at the front top corner (bridge side) of the backwall. This load is not applied, however, when designing the abutment wall (stem) or footing. Assuming an abutment length of 48 feet and a backwall width of 2.0 feet, the backwall live load is computed as follows:

$$R_{LL\ backwall} = \frac{(0.85)\left[(3\text{ lanes})\left(\frac{2\text{ wheels}}{\text{lane}}\right)\left(\frac{16\text{ kips}}{\text{wheel}}\right)(1.33)+(3\text{ lanes})(0.64\text{ klf})(2.0\text{ feet})\right]}{48\text{ feet}}$$

$$= 2.33\ \frac{K}{ft}$$

It should be noted that dynamic load allowance is applied to the truck live load only and not to the lane live load. This live load configuration on the abutment backwall is illustrated in Figure 12.8-2.

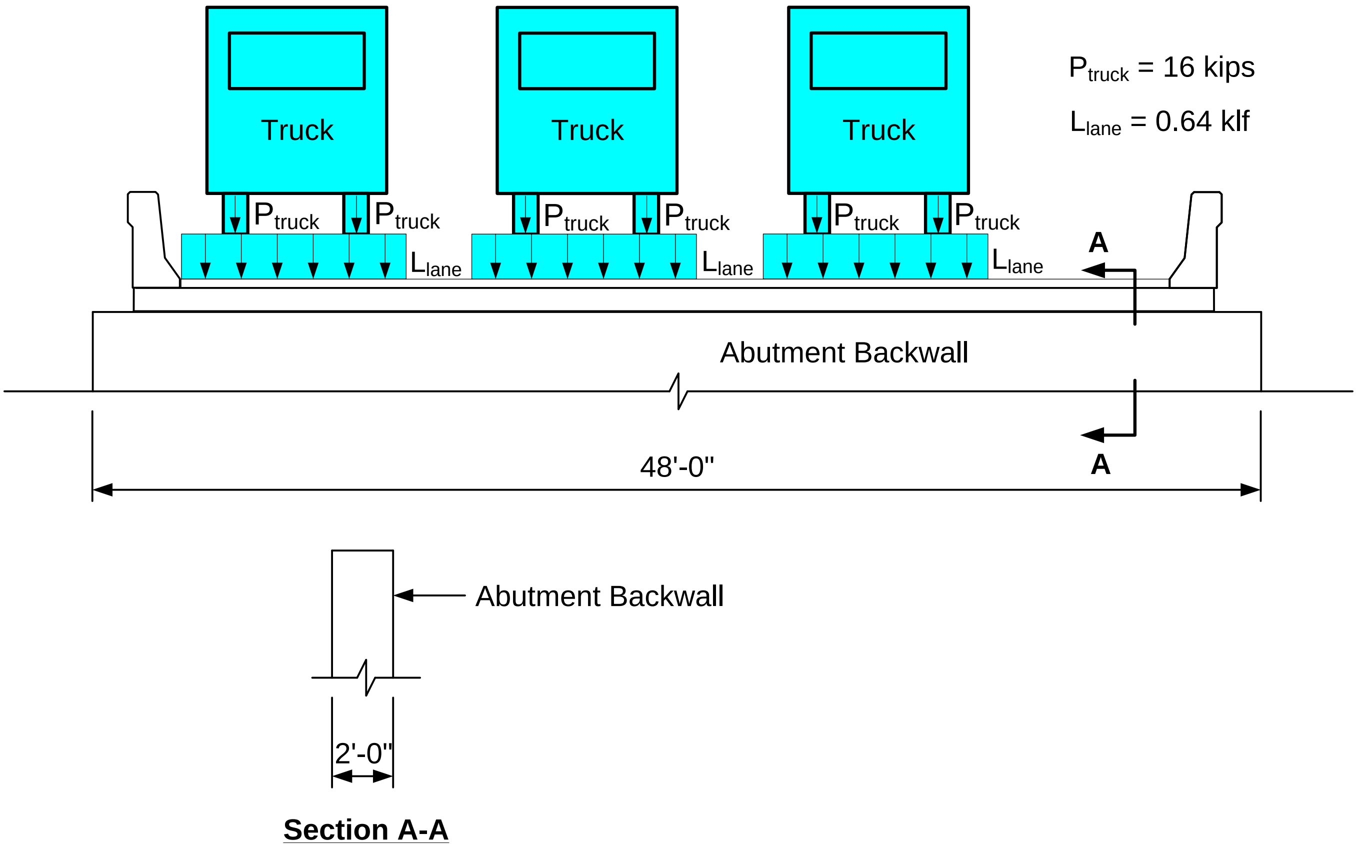


Figure 12.8-2
Live Load on Abutment Backwall

To compute the live loads applied to the abutment beam seat, the live load reactions should be obtained for one lane loaded using girder design software. For this example, for one design lane, the maximum truck live load reaction is 60.8 kips and the maximum lane live load reaction is 19.2 kips. In addition, assume that the abutment is relatively high; the load can therefore be distributed equally over the full length of the abutment. For wall (stem) design, the controlling maximum live loads applied at the beam seat are computed as follows, using three design lanes and using both dynamic load allowance and the multiple presence factor:

$$R_{LL\ stem} = \frac{(3\text{ lanes})(0.85)\left[(60.8\text{ kips})(1.33)+(19.2\text{ kips})\right]}{48\text{ feet}} = 5.32\ \frac{\text{K}}{\text{ft}}$$

This live load configuration for an abutment beam seat is illustrated in Figure 12.8-3.

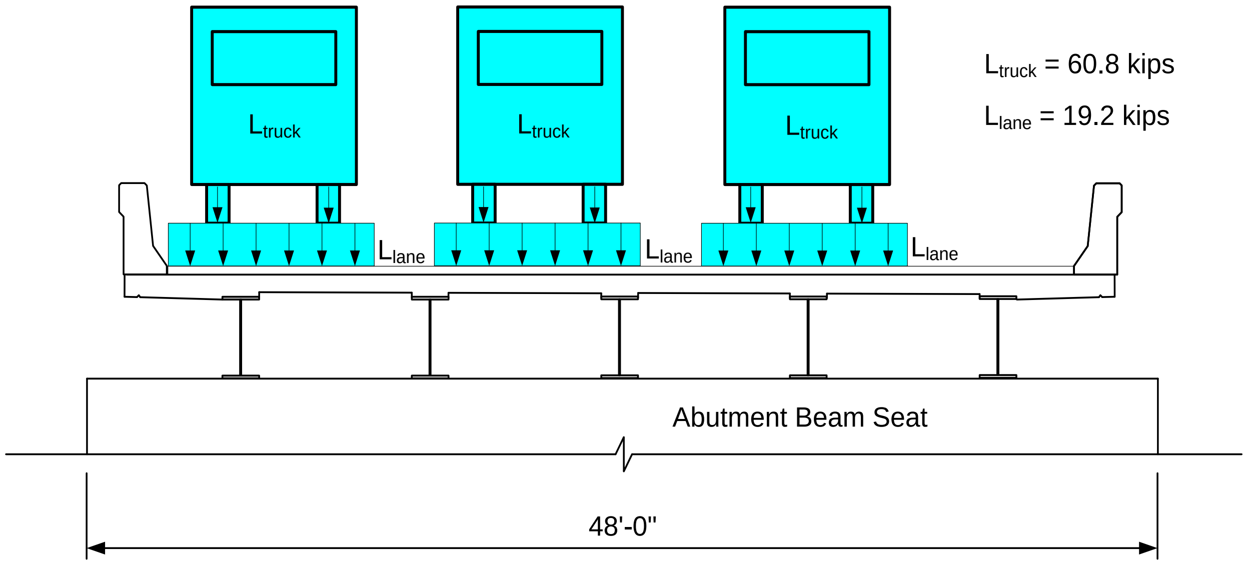


Figure 12.8-3
Live Load on Abutment Beam Seat

For a continuous bridge, the minimum live load applied to the abutment beam seat can be obtained based on the minimum (negative) live load reactions taken from girder design software output.

For footing design, the dynamic load allowance is not included. Therefore, the controlling maximum live loads applied at the beam seat are computed as follows:

$$R_{LL\ footing} = \frac{(3\ \text{lanes})(0.85)[60.8\ \text{kips} + 19.2\ \text{kips}]}{48\ \text{feet}} = 4.25\ \frac{K}{ft}$$

12.8.2 Load Modifiers and Load Factors

Table 12.8-1 presents the load modifiers used for abutment and wing wall design.

Description	Load Modifier
Ductility	1.00
Redundancy	1.00
Operational classification	1.00

Table 12.8-1
Load Modifiers Used in Abutment Design

Table 12.8-2 presents load factors used for abutment and wing wall design. Load factors presented in this table are based on the Strength I and Service I limit states. The load factors for WS and WL equal 0.00 for Strength I. Load factors for the Service I limit state for WS and WL are shown in the table below. Only apply these loads in the longitudinal direction.

Direction of Load	Specific Loading	Load Factor		
		Strength I		Service I
		Max.	Min.	
Load factors for vertical loads	Superstructure DC dead load	1.25	0.90	1.00
	Superstructure DW dead load	1.50	0.65	1.00
	Superstructure live load	1.75	1.75	1.00
	Approach slab dead load	1.25	0.90	1.00
	Approach slab live load	1.75	1.75	1.00
	Wheel loads located directly on the abutment backwall	1.75	1.75	1.00
	Earth surcharge	1.50	0.75	1.00
	Earth pressure	1.35	1.00	1.00
	Water load	1.00	1.00	1.00
	Live load surcharge	1.75	1.75	1.00
Load factors for horizontal loads	Substructure wind load, WS	0.00	0.00	0.00
	Superstructure wind load, WS	0.00	0.00	1.00
	Superstructure wind on LL, WL	0.00	0.00	1.00
	Vehicular braking force from live load	1.75	1.75	1.00
	Temperature and shrinkage*	1.20*	0.50*	1.00
	Earth pressure (active)	1.50	0.90	1.00
	Earth surcharge	1.50	0.75	1.00
	Live load surcharge	1.75	1.75	1.00

Table 12.8-2
Load Factors Used in Abutment Design

* Use the minimum load factor for temperature and shrinkage unless checking for deformations.

12.8.3 Live Load Surcharge

The equivalent heights of soil for vehicular loading on abutments perpendicular to traffic are as presented in **LRFD [Table 3.11.6.4-1]** and in Table 12.8-3. Values are presented for various abutment heights. The abutment height, as used in Table 12.8-3, is taken as the distance between the top surface of the backfill at the back face of the abutment and the bottom of the footing along the pressure surface being considered. Linear interpolation should be used for intermediate abutment heights. The load factors for both vertical and horizontal components of live load surcharge are as specified in **LRFD [Table 3.4.1-1]** and in Table 12.8-2.

Abutment Height (Feet)	h_{eq} (Feet)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Table 12.8-3
Equivalent Height, h_{eq}, of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

WisDOT policy item:

The equivalent height of soil for vehicular loading on retaining walls parallel to traffic shall be 2.0 feet, regardless of the wall height. For standard unit weight of soil equal to 120 pcf, the resulting live load surcharge is 240 psf.

For abutments without reinforced concrete approaches, the equivalent height of soil for vehicular loading on abutments shall be based on Table 12.8-3. For abutments with reinforced concrete approaches, one half of the equivalent height of soil shall be used to calculate the horizontal load on the abutment.

12.8.4 Other Abutment Design Parameters

The equivalent fluid unit weights of soils are as presented in **LRFD [Table 3.11.5.5-1]**. Values are presented for loose sand or gravel, medium dense sand or gravel, and dense sand or gravel. Values are also presented for level or sloped backfill and for at-rest or active soil conditions.

Table 12.8-4 presents other parameters used in the design of abutments and wing walls. Standard details are based on the values presented in Table 12.8-4.

Description	Value
Bottom reinforcing steel cover	3.0 inches
Top reinforcing steel cover	2.0 inches
Unit weight of concrete	150 pcf
Concrete strength, f'_c	3.5 ksi
Reinforcing steel yield strength, f_y	60 ksi
Reinforcing steel modulus of elasticity, E_s	29,000 ksi
Unit weight of soil	120 pcf
Unit weight of structural backfill	120 pcf
Soil friction angle	30 degrees

Table 12.8-4
Other Parameters Used in Abutment Design

12.8.5 Abutment and Wing Wall Design in Wisconsin

The standard details for abutments and wing walls were developed as an envelope of the loading conditions produced by the standard superstructure types, span lengths and geometric conditions presented in this manual. Prior BOS approval is required and special consideration should be given to designs that are outside of the limits presented in the standard details. The loading conditions, material properties and design methods presented in this chapter should be used for these special designs.

WisDOT policy items:

The resistance of the wing pile to horizontal forces should not be included in the calculations for the wing capacity.

The passive earth resistance can only be developed if there is significant movement of the wing. The soil under the wing may settle or otherwise erode. Therefore, the resistance of the soil friction and the passive earth pressure should not be utilized in resisting the forces on wing walls.

In computing the weight of the approach slab, assume there is settlement under the approach slab and place one-half of the weight of the slab on the abutment. An unfactored dead load value of 1.2 klf shall be used for concrete approach slabs and 2.0 klf for structural approach slabs. An unfactored live load value of 0.900 klf shall be applied to abutment approach slabs when used. Approach reactions shall act along the centroid of the foundation.

The dynamic load allowance shall be applied to the live load for all abutment elements located above the ground line per **LRFD [3.6.2]**.

12.8.6 Horizontal Pile Resistance

The following procedure shall be used to verify the horizontal resistance of the piles for A3 and A4 abutments.

Given information:

Horizontal Loads	Unfactored (klf)		Load Factor		Factored Load (klf)
Earth Pressure	5.5	x	1.50	=	8.25
Live Load Surcharge	1.0	x	1.75	=	1.75
Temp. Load from Bearings	0.6	x	0.50	=	0.30
			Total, Hu	=	10.3

Back row pile spacing =	8.0 feet
Front row pile spacing =	5.75 feet
Ultimate Vertical Resistance, 12 3/4" CIP, Pr =	210 kips per pile
Factored Vertical Load on Front Row Pile*	160 kips per pile
Ultimate Horizontal Resistance of back row pile (from Geotech Report), Hr =	14 kips per pile
Ultimate Horizontal Resistance of front row pile (from Geotech Report), Hr =	11 kips per pile

* When calculating the horizontal component of the battered pile, use the actual factored load on the pile resulting from the loading conditions where the horizontal loads are maximized and the vertical loads are minimized.

Calculate horizontal component of the battered pile. The standard pile batter is 1:4.

$$Hr_{battered} = 160\left(\frac{1}{\sqrt{1^2 + 4^2}}\right)$$

$Hr_{battered}$ = 38.8 kips per pile

Calculate ultimate resistance provided by the pile configuration:

$$Hr = \left(\frac{14}{8.0}\right) + \left(\frac{11}{5.75}\right) + \left(\frac{38.8}{5.75}\right)$$

Hr = 10.4 klf

Hr > Hu = 10.3 klf OK

12.9 Abutment Body Details

There are many different body sections that are utilized for each of the different abutment types. When designing these sections, it is inadvisable to use small and highly reinforced sections. As a general principle, it is better to use a lot of concrete and less reinforcing steel, thus making parts relatively massive and stiff. Adequate horizontal reinforcement and vertical contraction joints are essential to prevent cracking, especially when wing walls are poured monolithically with the abutment body.

The bottom of abutment bodies are normally constructed on a horizontal surface. However, abutments constructed on a horizontal surface may require one end of the body to be much higher than the opposite end due to the vertical geometry of the bridge. This sometimes requires an extremely long and high wing wall. For these extreme cases, the bottom of the abutment body can be stepped.

The berm in front of the body is held level even though the body is stepped. A minimum distance of 2.5 feet between the top of berm and the top of beam seat is allowed. Minimum ground cover as shown in the Standard Detail for Abutments must be maintained.

Stepping the bottom of the body may result in a longer bridge. This is usually more costly than holding the body level and using larger wings and beam seats. Stepped abutments are also more difficult to build. Engineering judgment must be exercised when determining if the bottom of the abutment should be level or stepped. Generally, if a standard wing wall design cannot be used, the bottom of the abutment body should be stepped.

12.9.1 Construction Joints

In a U-shaped abutment with no joint between the wings and the body, traffic tends to compact the fill against the three sides of the abutment. When the temperature drops, the abutment body concrete cannot shrink without tending to squeeze the warmer fill inside. The resistance of the fill usually exceeds the tensile or shearing strength of the body or wing, and cracks result.

If contraction joints are not provided in long abutment bodies, nature usually creates them. To prevent uncontrolled cracking in the body or cracking at the body-wing joint, body pours are limited to a maximum of 50 feet. Expansion joints are required at a maximum of 90 feet, as specified in **LRFD [11.6.1.6]**.

WisDOT exception to AASHTO:

LRFD [11.6.1.6] specifies that contraction joints shall be provided at intervals not exceeding 30 feet for conventional retaining walls and abutments. However, WisDOT has not experienced significant problems with 50 feet and uses a maximum interval of 50 feet.

Shear keys are provided in construction joints to allow the center pour to maintain the beneficial stabilizing effects from the wings. The shear keys enable the end pours, with their counterfort action due to the attached wing, to provide additional stability to the center pour. Reinforcing steel should be extended through the joint.

In general, body construction joints are keyed to hold the parts in line. Water barriers are used to prevent leakage and staining. Steel girder superstructures generally permit a small movement at construction joints without cracking the concrete slab. In the case of concrete slab or prestressed concrete girder construction, a crack will frequently develop in the deck above the abutment construction joint. The designer should consider this when locating the construction joint.

12.9.2 Beam Seats

Because of the bridge deck cross-slopes and/or skewed abutments, it is necessary to provide beam seats of different elevations on the abutment. The tops of these beam seats are poured to the plan elevations and are made level except when elastomeric bearing pads are used and grades are equal to or exceed 1%. For this case, the beam seat should be parallel to the bottom of girder or slab. Construction tolerances make it difficult to obtain the exact beam seat elevation.

When detailing abutments, the differences in elevations between adjacent beam seats are provided by sloping the top of the abutment between level beam seats. For steel girders, the calculation of beam seat elevations and use of shim plates at abutments, to account for thicker flanges substituted for plan flange thickness, is described on the Standard *Plate Girder Details* in Chapter 24.

See the abutment standards for additional reinforcing required when beam seats are 4” higher, or more, than the lowest beam seat.

12.10 Timber Abutments

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d galvanized nails to timber nailing strips which are bolted to the piling, or between the flanges of “HP” piles.

12.11 Bridge Approach Design and Construction Practices

While most bridge approaches are reasonably smooth and require a minimum amount of maintenance, there are also rough bridge approaches with maintenance requirements. The bridge designer should be aware of design and construction practices that minimize bridge approach maintenance issues. Soils, design, construction and maintenance engineers must work together and are jointly responsible for efforts to eliminate rough bridge approaches.

An investigation of the foundation site is important for bridge design and construction. The soils engineer, using tentative grades and foundation site information, can provide advice on the depth of material to be removed, special embankment foundation drainage, surcharge heights, waiting periods, construction rates and the amount of post-construction settlement that can be anticipated. Some typical bridge approach problems include the following:

- Settlement of pavement at end of approach slab
- Uplift of approach slab at abutment caused from swelling soils or freezing
- Backfill settlement under flexible pavement
- Approach slab not adequately supported at the abutments
- Erosion due to water infiltration

Most bridge approach problems can be minimized during design and construction by considering the following:

- Embankment height, material and construction methods
- Subgrade, subbase and base material
- Drainage-runoff from bridge, surface drains and drainage channels
- Special approach slabs allowing for pavement expansion

Post-construction consolidation of material within the embankment foundation is the primary contributor to rough bridge approaches. Soils which consist predominantly of sands and gravels are least susceptible to consolidation and settlement. Soils with large amounts of shales, silts and plastic clays are highly susceptible to consolidation.

The following construction measures can be used to stabilize foundation materials:

- Consolidate the natural material. Allow sufficient time for consolidation under the load of the embankment. When site investigations indicate an excessive length of time is required, other courses of corrective action are available. Use of a surcharge fill is effective where the compressive stratum is relatively thin and sufficient time is available for consolidation.

- Remove the material either completely or partially. This procedure is practical if the foundation depth is less than 15 feet and above the water table.

- Use lightweight embankment materials. Lightweight materials (fly ash, expanded shale and cinders) have been used with apparent success for abutment embankment construction to lessen the load on the foundation materials.

Abutment backfill practices that help minimize either settlement or swell include the following:

- Use of select materials

- Placement of relatively thin 4- to 6-inch layers

- Strict control of moisture and density

- Proper compaction

- Installation of moisture barriers

It is generally recognized by highway and bridge engineers that bridge abutments cause relatively few of the problems associated with bridge approaches. Proper drainage needs to be provided to prevent erosion of embankment or subgrade material that could cause settlement of the bridge approach. It is essential to provide for the removal of surface water that leaks into the area behind the abutment by using weepholes and/or drain tile. In addition, water infiltration between the approach slab and abutment body and wings must be prevented.

Reinforced concrete approach slabs are the most effective means for controlling surface irregularities caused by settlement. It is also important to allow enough expansion movement between the approach slab and the approach pavement to prevent horizontal thrust on the abutment.

The bridge designer should determine if a structural approach slab is required and coordinate details with the roadway engineer. Usage of structural approach slabs is currently based on road functional classifications and considerations to traffic volumes (AADT), design speeds, and settlement susceptibility.

WisDOT policy item:

Structural approach slabs shall be used on all bridges carrying traffic volumes greater than 3500 AADT in the future design year. Structural approach slabs are not required on buried structures and culverts and should not be used on rehabilitation projects. Other locations can be considered with the approval of the Chief Structural Design Engineer.

Standards for Structural Approach Slab for Type A1, A3, and A4 Abutments and Structural Approach Slab Details for Type A1, A3, and A4 Abutments are available for guidance.

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Table of Contents

13.1 General

Piers are an integral part of the load path between the superstructure and the foundation. Piers are designed to resist the vertical loads from the superstructure, as well as the horizontal superstructure loads not resisted by the abutments. The magnitude of the superstructure loads applied to each pier shall consider the configuration of the fixed and expansion bearings, the bearing types and the relative stiffness of all of the piers. The analysis to determine the horizontal loads applied at each pier must consider the entire system of piers and abutments and not just the individual pier. The piers shall also resist loads applied directly to them, such as wind loads, ice loads, water pressures and vehicle impact.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

WisDOT policy item:

At this time, evaluation and plan preparations for accommodating a noted allowance for a precast pier option as indicated in 7.1.4.1.2 is only required for I-39/90 Project bridges. All other locations statewide may consider providing a noted allowance for a precast option. Contact the Bureau of Structures Development Section for further guidance.

13.1.1 Pier Type and Configuration

Many factors are considered when selecting a pier type and configuration. The engineer should consider the superstructure type, the characteristics of the feature crossed, span lengths, bridge width, bearing type and width, skew, required vertical and horizontal clearance, required pier height, aesthetics and economy. For bridges over waterways, the pier location relative to the floodplain and scour sensitive regions shall also be considered.

The connection between the pier and superstructure is usually a fixed or expansion bearing which allows rotation in the longitudinal direction of the superstructure. This has the effect of eliminating longitudinal moment transfer between the superstructure and the pier. In rare cases when the pier is integral with the superstructure, this longitudinal rotation is restrained and moment transfer between the superstructure and the pier occurs. Pier types illustrated in the Standard Details shall be considered to be a pinned connection to the superstructure.

On grades greater than 2 percent, the superstructure tends to move downhill towards the abutment. The low end abutment should be designed as fixed and the expansion joint or joints placed on the uphill side or high end abutment. Consideration should also be given to fixing more piers than a typical bridge on a flat grade.

13.1.2 Bottom of Footing Elevation

The bottom of footing elevation for piers outside of the floodplain is to be a minimum of 4' below finished ground line unless the footings are founded on solid rock. This requirement is intended to place the bottom of the footing below the frost line.

A minimum thickness of 2'-0" shall be used for spread footings and 2'-6" for pile-supported footings. Spread footings are permitted in streams only if they are founded on rock. Pile cap footings are allowed above the ultimate scour depth elevation if the piling is designed assuming the full scour depth condition.

The bottom of footing elevation for pile cap footings in the floodplain is to be a minimum of 6' below stable streambed elevation. Stable streambed elevation is the normal low streambed elevation at a given pier location when not under scour conditions. When a pile cap footing in the floodplain is placed on a concrete seal, the bottom of footing is to be a minimum of 4' below stable streambed elevation. The bottom of concrete seal elevation is to be a minimum of 8' below stable streambed elevation. These requirements are intended to guard against the effects of scour.

13.1.3 Pier Construction

Except for pile encased piers (see Standard for Pile Encased Pier) and seal concrete for footings, all footing and pier concrete shall be placed in the dry. Successful underwater concreting requires special concrete mixes, additives and placement procedures, and the risk of error is high. A major concern in underwater concreting is that the water in which the concrete is placed will wash away cement and sand, or mix with the concrete, and increase the water-to-cement ratio. It was previously believed that if the lower end of the tremie is kept immersed in concrete during a placement, then the new concrete flows under and is protected by previously placed concrete. However, tests performed at the University of California at Berkeley show that concrete exiting a tremie pipe may exhibit many different flow patterns exposing more concrete to water than expected. A layer of soft, weak and water-laden mortar called laitance may also form within the pour. Slump tests do not measure shear resistance, which is the best predictor of how concrete will flow after exiting a tremie pipe.

Footing excavation adjacent to railroad tracks which falls within the critical zone shown on Standard for Highway Over Railroad Design Requirements requires an approved shoring system. Excavation, shoring and cofferdam costs shall be considered when evaluating estimated costs for pier construction, where applicable. Erosion protection is required for all excavations.

13.2 Pier Types

The pier types most frequently used in Wisconsin are:

- Multi-column piers (Standards for Multi-Columned Pier and for Multi-Columned Pier – Type 2)
- Pile bents (Standard for Pile Bent)
- Pile encased piers (Standard for Pile Encased Pier)
- Solid single shaft / hammerheads (Standards for Hammerhead Pier and for Hammerhead Pier – Type 2)

Design loads shall be calculated and applied to the pier in accordance with 13.4 and 13.5. The following sections discuss requirements specific to each of the four common pier types.

13.2.1 Multi-Column Piers

Multi-column piers, as shown in Standard for Multi-Columned Pier, are the most commonly used pier type for grade separation structures. Refer to 13.6 for analysis guidelines.

A minimum of three columns shall be provided to ensure redundancy should a vehicular collision occur. If the pier cap cantilevers over the outside columns, a square end treatment is preferred over a rounded end treatment for constructability. WisDOT has traditionally used round columns. Column spacing for this pier type is limited to a maximum of 25'.

Multi-column piers are also used for stream crossings. They are especially suitable where a long pier is required to provide support for a wide bridge or for a bridge with a severe skew angle.

Continuous or isolated footings may be specified for multi-column piers. The engineer should determine estimated costs for both footing configurations and choose the more economical configuration. Where the clear distance between isolated footings would be less than 4'-6", a continuous footing shall be specified.

A variation of the multi-column pier in Standard for Multi-Columned Pier is produced by omitting the cap and placing a column under each girder. This detail has been used for steel girders with girder spacing greater than 12'. This configuration is treated as a series of single column piers. The engineer shall consider any additional forces that may be induced in the superstructure cross frames at the pier if the pier cap is eliminated. The pier cap may not be eliminated for piers in the floodplain, or for continuous slab structures which need the cap to facilitate replacement of the slab during future rehabilitation.

See Standard for Highway Over Railroad Design Requirements for further details on piers supporting bridges over railways.

13.2.2 Pile Bents

Pile bents are most commonly used for small to intermediate stream crossings and are shown on the Standard for Pile Bent.

Pile bents shall not be used to support structures over roadways or railroads due to their susceptibility to severe damage should a vehicular collision occur.

For pile bents, pile sections shall be limited to 12¾" or 14" diameter cast-in-place reinforced concrete piles with steel shells spaced at a minimum center-to-center spacing of 3'. A minimum of five piles per pier shall be used on pile bents. When a satisfactory design cannot be developed with one of these pile sections at the minimum spacing, another pier type should be selected. The outside piles shall be battered 2" per foot, and the inside piles shall be driven vertically. WisDOT does not rely on the shell of CIP piles for capacity; therefore these piles are less of a concern for long term reduced capacity due to corrosion than steel H-piles. For that reason the BOS Development Chief must give approval for the use of steel H-piles in open pile bents.

Because of the minimum pile spacing, the superstructure type used with pile bents is generally limited to cast-in-place concrete slabs, prestressed girders and steel girders with spans under approx. 70' and precast, prestressed box girders less than 21" in height.

To ensure that pile bents are capable of resisting the lateral forces resulting from floating ice and debris or expanding ice, the maximum distance from the top of the pier cap to the stable streambed elevation, including scour, is limited to:

- 15' for 12¾" diameter piles (or 12" H-piles if exception is granted).
- 20' for 14" diameter piles (or 14" H-piles if exception is granted).

Use of the pile values in Table 11.3-5 or Standard for Pile Details is valid for open pile bents due to the exposed portion of the pile being inspectable.

The minimum longitudinal reinforcing steel in cast-in-place piles with steel shells is 6-#7 bars in 12" piles and 8-#7 bars in 14" piles. The piles are designed as columns fixed from rotation in the plane of the pier at the top and at some point below streambed.

All bearings supporting a superstructure utilizing pile bents shall be fixed bearings or semi-expansion.

Pile bents shall meet the following criteria:

- If the water velocity, Q_{100}, is greater than 7 ft/sec, the quantity of the 100-year flood shall be less than 12,000 ft^3/sec.
- If the streambed consists of unstable material, the velocity of the 100-year flood shall not exceed 9 ft/sec.

Pile bents may only be specified where the structure is located within Area 3, as shown in the *Facilities Development Manual 13-1-15*, *Attachment 15.1* and where the piles are not exposed to water with characteristics that are likely to cause accelerated corrosion.

The minimum cap size shall be 3' wide by 3'-6" deep and the piles shall be embedded into the cap a minimum of 2'-0.

13.2.3 Pile Encased Piers

Pile encased piers are similar to pile bents except that a concrete encasement wall surrounds the piles. They are most commonly used for small to intermediate stream crossings where a pile bent pier is not feasible. Pile encased piers are detailed on Standard for Pile Encased Pier.

An advantage of this pier type is that the concrete encasement wall provides greater resistance to lateral forces than a pile bent. Also the hydraulic characteristics of this pier type are superior to pile bents, resulting in a smoother flow and reducing the susceptibility of the pier to scour at high water velocities. Another advantage is that floating debris and ice are less likely to accumulate against a pile encased pier than between the piles of a pile bent. Debris and ice accumulation are detrimental because of the increased stream force they induce. In addition, debris and ice accumulation cause turbulence at the pile, which can have the effect of increasing the local scour potential.

Pile sections shall be limited to 10", 12" or 14" steel HP piles, or 10¾", 12¾" or 14" diameter cast-in-place concrete piles with steel shells. Minimum center-to-center spacing is 3'. Where difficult driving conditions are expected, oil field pipe may be specified in the design. A minimum of five piles per pier shall be used. When a satisfactory design cannot be developed with one of these pile sections at the minimum spacing, another pier type should be selected. The inside and outside piles shall be driven vertically.

In most cases, a cofferdam should be used for pile encased piers. See 13.11.5 for additional guidance regarding cofferdams. Total pier height shall be less than 25 feet.

All bearings supporting a superstructure utilizing pile encased piers shall be fixed bearings or semi-expansion.

The connection between the superstructure and the pier shall be designed to transmit the portion of the superstructure design loads assumed to be taken by the pier.

The concrete wall shall be a minimum of 2'-6" thick. The top 3' of the wall is made wider if a larger bearing area is required. See Standard for Pile Encased Pier for details. The bottom of the wall shall be placed 2' to 4' below stable streambed elevation, depending upon stream velocities and frost depth.

13.2.4 Solid Single Shaft / Hammerheads

Solid single shaft piers are used for all types of crossings and are detailed on Standards for Hammerhead Pier and for Hammerhead Pier – Type 2. The choice between using a multi-column pier and a solid single shaft pier is based on economics and aesthetics. For high level bridges, a solid single shaft pier is generally the most economical and attractive pier type available.

The massiveness of this pier type provides a large lateral load capacity to resist the somewhat unpredictable forces from floating ice, debris and expanding ice. They are suitable for use on major rivers adjacent to shipping channels without additional pier protection. When used adjacent to railroad tracks, crash walls are not required.

If a cofferdam is required and the upper portion of a single shaft pier extends over the cofferdam, an optional construction joint is provided 2’ above the normal water elevation. Since the cofferdam sheet piling is removed by extracting vertically, any overhead obstruction prevents removal and this optional construction joint allows the contractor to remove sheet piling before proceeding with construction of the overhanging portions of the pier.

A hammerhead pier shall not be used when the junction between the cap and the shaft would be less than the cap depth above normal water. Hammerhead piers are not considered aesthetically pleasing when the shaft exposure above water is not significant. A feasible alternative in this situation would be a wall type solid single shaft pier or a multi-column pier. On a wall type pier, both the sides and ends may be sloped if desired, and either a round, square or angled end treatment is acceptable. If placed in a waterway, a square end type is less desirable than a round or angled end.

13.2.5 Aesthetics

Refer to Chapter 4 for additional information about aesthetics.

13.3 Location

Piers shall be located to provide a minimum interference to flood flow. In general, place the piers parallel with the direction of flood flow. Make adequate provision for drift and ice by increasing span lengths and vertical clearances, and by selecting proper pier types. Special precautions against scour are required in unstable streambeds. Navigational clearance shall be considered when placing piers for bridges over navigable waterways. Coordination with the engineer performing the hydraulic analysis is required to ensure the design freeboard is met, the potential for scour is considered, the hydraulic opening is maintained and the flood elevations are not adversely affected upstream or downstream. Refer to Chapter 8 for further details.

In the case of railroad and highway separation structures, the spacing and location of piers and abutments is usually controlled by the minimum horizontal and vertical clearances required for the roadway or the railroad. Other factors such as utilities or environmental concerns may influence the location of the piers. Sight distance can impact the horizontal clearance required for bridges crossing roadways on horizontally curved alignments. Requirements for vertical and horizontal clearances are specified in Chapter 3 – Design Criteria. Crash wall requirements are provided on Standard for Highway Over Railroad Design Requirements.

Cost may also influence the number of piers, and therefore the number of spans, used in final design. During the planning stages, an analysis should be performed to determine the most economical configuration of span lengths versus number of piers that meet all of the bridge site criteria.

13.4 Loads on Piers

The following loads shall be considered in the design of piers. Also see 13.5 for additional guidance regarding load application.

13.4.1 Dead Loads

The dead load forces, DC and DW, acting on the piers shall include reactions from the superstructure. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. The pier diaphragm weight may be applied through the girders. Different load factors are applied to each of these dead load types.

For a detailed discussion of the application of dead load, refer to 17.2.4.1.

13.4.2 Live Loads

The HL-93 live load shall be used for all new bridge designs and is placed in 12'-wide design lanes. If fewer lane loads are used than what the roadway width can accommodate, the loads shall be kept within their design lanes. The design lanes shall be positioned between the curbs, ignoring shoulders and medians, to maximize the effect being considered. Refer to 17.2.4.2 for a detailed description of the HL-93 live load. For pier design, particular attention should be given to the double truck load described in 17.2.4.2.4. This condition places two trucks, spaced a minimum of 50' apart, within one design lane and will often govern the maximum vertical reaction at the pier.

WisDOT policy items:

A 10 foot design lane width may be used for the distribution of live loads to a pier cap.

The dynamic load allowance shall be applied to the live load for all pier elements located above the ground line per **LRFD [3.6.2]**.

For girder type superstructures, the loads are transmitted to the pier through the girders. For pier design, simple beam distribution is used to distribute the live loads to the girders. The wheel and lane loads are therefore transversely distributed to the girders by the lever rule as opposed to the Distribution Factor Method specified in **LRFD [4.6.2.2.2]**. The lever rule linearly distributes a portion of the wheel load to a particular girder based upon the girder spacing and the distance from the girder to the wheel load. The skew of the structure is not considered when calculating these girder reactions. Refer to 17.2.10 for additional information about live load distribution to the substructure and to Figure 17.2-17 for application of the lever rule.

For slab type superstructures, the loads are assumed to be transmitted directly to the pier without any transverse distribution. This assumption is used even if the pier cap is not integral with the superstructure. The HL-93 live load is applied as concentrated wheel loads combined with a uniform lane load. The skew of the structure is considered when applying these loads to the cap. The lane width is then divided by the cosine of the skew angle, and the load is distributed over the new lane width along the pier centerline.

As a reminder, the live load force to the pier for a continuous bridge is based on the *reaction*, not the sum of the adjacent span shear values. A pier beneath non-continuous spans (at an expansion joint) uses the sum of the reactions from the adjacent spans.

13.4.3 Vehicular Braking Force

Vehicular braking force, BR, is specified in **LRFD [3.6.4]** and is taken as the greater of:

- 25% of the axle loads of the design truck
- 25% of the axle loads of the design tandem
- 5% of the design truck plus lane load
- 5% of the design tandem plus lane load

The loads applied are based on loading one-half the adjacent spans. Do not use a percentage of the live load reaction. All piers receive this load. It is assumed that the braking force will be less than the dead load times the bearing friction value and all force will be transmitted to the given pier. The tandem load, even though weighing less than the design truck, must be considered for shorter spans since not all of the axles of the design truck may be able to fit on the tributary bridge length.

This force represents the forces induced by vehicles braking and may act in all design lanes. The braking force shall assume that traffic is traveling in the same direction for all design lanes as the existing lanes may become unidirectional in the future. This force acts 6' above the bridge deck, but the longitudinal component shall be applied at the bearings. It is not possible to transfer the bending moment of the longitudinal component acting above the bearings on typical bridge structures. The multiple presence factors given by **LRFD [3.6.1.1.2]** shall be considered. Per **LRFD [3.6.2.1]**, the dynamic load allowance shall not be considered when calculating the vehicular braking force.

13.4.4 Wind Loads

WisDOT exception to AASHTO:

The design wind velocity, V_{DZ}, from **LRFD [3.8.1.1]** shall be set to 100 mph for all bridge elevations.

In 13.4.4.1 and 13.4.4.2, the base wind pressure, P_B, will not be modified based on the elevation of the bridge and shall be taken as:

$$P_D = P_B$$

Where:

P_D = Design wind pressure at all elevations (ksf)

Wind loads are divided into the following four types.

13.4.4.1 Wind Load on Superstructure

To determine WS, the base wind pressures, P_B, presented in Table 13.4-1 shall be applied to the superstructure elements as specified in **LRFD [3.8.1.2.2]**.

Wind Skew Angle (deg.)	Trusses, Columns and Arches		Girders	
	Lateral Load (ksf)	Longitudinal Load (ksf)	Lateral Load (ksf)	Longitudinal Load (ksf)
0	0.075	0.000	0.050	0.000
15	0.070	0.012	0.044	0.006
30	0.065	0.028	0.041	0.012
45	0.047	0.041	0.033	0.016
60	0.024	0.050	0.017	0.019

Table 13.4-1
Superstructure Base Wind Pressures

The wind skew angle shall be taken as measured from a perpendicular to the longitudinal axis. The wind direction used shall be that which produces the maximum force effects on the member. Transverse and longitudinal pressures shall be applied simultaneously. The longitudinal component shall be applied at the bearing elevation, and the transverse component shall be applied at its actual elevation.

WisDOT policy item:

The following conservative values for wind on superstructure, WS, may be used for all girder bridges:

- 0.05 ksf, transverse
- 0.012 ksf, longitudinal

Both forces shall be applied simultaneously. Do not apply to open rails or fences. Do apply this force to all parapets, including parapets located between the roadway and sidewalk if there is an open rail or fence on the edge of the sidewalk.

13.4.4.2 Wind Load Applied Directly to Substructure

To determine WS for wind applied directly to substructures, the base wind pressure, P_B, to be applied to the substructure units is 0.040 ksf as specified in **LRFD [3.8.1.2.3]**. This load can be resolved into components based on skew, or the following policy item can be followed:

WisDOT policy item:

The following values for wind applied directly to substructures, WS, may be used for all bridges:

- 0.040 ksf, transverse (along axis of substructure unit)
- 0.040 ksf, longitudinal (normal to axis of substructure unit)

Both forces shall be applied simultaneously.

13.4.4.3 Wind Load on Vehicles

As specified in **LRFD [3.8.1.3]** the wind force on vehicles, WL, is applied 6 ft. above the roadway. The longitudinal component shall be applied at the bearing elevation, and the transverse component shall be applied at its actual elevation.

WisDOT policy item:

The following values for wind on live load, WL, may be used for all bridges:

- 0.100 klf, transverse
- 0.040 klf, longitudinal

Both forces shall be applied simultaneously.

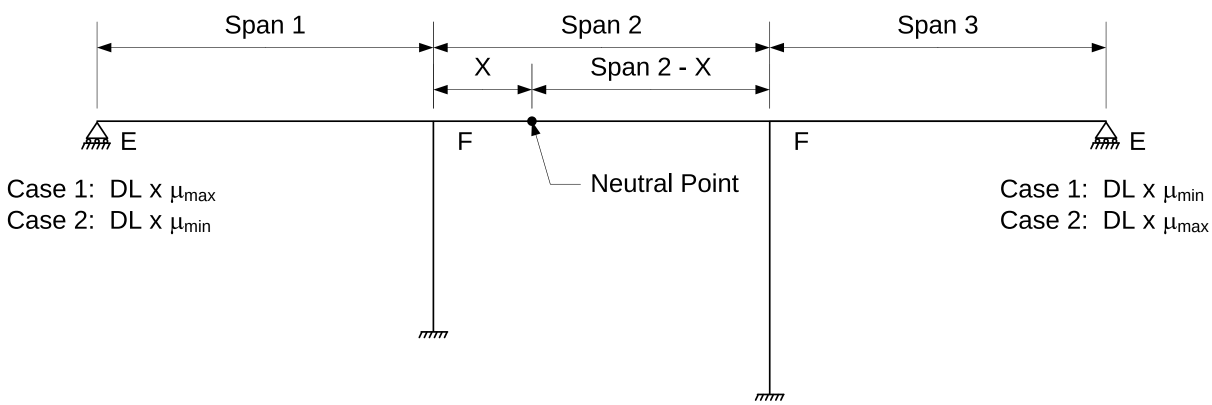
13.4.4.4 Vertical Wind Load

As specified in **LRFD [3.8.2]** an overturning vertical wind force, WS, shall be applied to limit states that do not involve wind on live load. A vertical upward wind force of 0.020 ksf times the out-to-out width of the bridge deck shall be considered a longitudinal line load. This lineal force shall be applied at the windward ¼ point of the deck, which causes the largest upward force at the windward fascia girder.

13.4.5 Uniform Temperature Forces

Temperature changes in the superstructure cause it to expand and contract along its longitudinal axis. These length changes induce forces in the substructure units based upon the fixity of the bearings, as well as the location and number of substructure units. The skew angle of the pier shall be considered when determining the temperature force components.

In determining the temperature forces, TU, applied to each substructure unit, the entire bridge superstructure length between expansion joints is considered. In all cases, there is a neutral point on the superstructure which does not move due to temperature changes. All temperature movements will then emanate outwards or inwards from this neutral point. This point is determined by assuming a neutral point. The sum of the expansion forces and fixed pier forces on one side of the assumed neutral point is then equated to the sum of the expansion forces and fixed pier forces on the other side of the assumed neutral point. Maximum friction coefficients are assumed for expansion bearings on one side of the assumed neutral point and minimum coefficients are assumed on the other side to produce the greatest unbalanced force for the fixed pier(s) on one side of the assumed neutral point. The maximum and minimum coefficients are then reversed to produce the greatest unbalanced force for the pier(s) on the other side of the assumed neutral point. For semi-expansion abutments, the assumed minimum friction coefficient is 0.06 and the maximum is 0.10. For laminated elastomeric bearings, the force transmitted to the pier is the shear force generated in the bearing due to temperature movement. Example E27-1.8 illustrates the calculation of this force. Other expansion bearing values can be found in Chapter 27 – Bearings. When writing the equation to balance forces, one can set the distance from the fixed pier immediately to one side of the assumed neutral point as ‘X’ and the fixed pier immediately to the other side as (Span Length – ‘X’). This is illustrated in Figure 13.4-1.



$$\text{Case1: } \sum DL \times \mu_{max} + \sum \frac{3EI\alpha TX}{144h^3} = \sum \frac{3EI\alpha T(\text{Span}\,2 - X)}{144h^3} + \sum DL \times \mu_{min}$$

(Lt. Abutment) (Pier 1) (Pier 2) (Rt. Abutment)

Figure 13.4-1
Neutral Point Location with Multiple Fixed Piers

As used in Figure 13.4-1:

E	=	Column or shaft modulus of elasticity (ksi)
I	=	Column or shaft gross moment of inertia about longitudinal axis of the pier (in^4)
α	=	Superstructure coefficient of thermal expansion (ft/ft/°F)

T = Temperature change of superstructure (°F)

μ = Coefficient of friction of the expansion bearing (dimensionless)

h = Column height (ft)

DL = Total girder dead load reaction at the bearing (kips)

X = Distance between the fixed pier and the neutral point (ft)

The temperature force on a single fixed pier in a bridge is the resultant of the unbalanced forces acting on the substructure units. Maximum friction coefficients are assumed for expansion bearings on one side of the pier and minimum coefficients are assumed on the other side to produce the greatest unbalanced force on the fixed pier. For bridges with only one pier (fixed), do not include temperature force, TU, in the design of the pier when the abutments are either fixed or semi-expansion.

The temperature changes in superstructure length are assumed to be along the longitudinal axis of the superstructure regardless of the substructure skew angle. This assumption is more valid for steel structures than for concrete structures.

The force on a column with a fixed bearing due to a temperature change in length of the superstructure is:

$$F = \frac{3EI\alpha TL}{144h^3}$$

Where:

L = Superstructure expansion length between neutral point and location being considered (ft)

F = Force per column applied at the bearing elevation (kips)

This force shall be resolved into components along both the longitudinal and transverse axes of the pier.

The values for computing temperature forces in Table 13.4-2 shall be used on Wisconsin bridges. Do not confuse this temperature change with the temperature range used for expansion joint design.

	Reinforced Concrete	Steel
Temperature Change	45 °F	90 °F
Coefficient of Thermal Expansion	0.0000060/°F	0.0000065/°F

Table 13.4-2
Temperature Expansion Values

Temperature forces on bridges with two or more fixed piers are based on the movement of the superstructure along its centerline. These forces are assumed to act normal and parallel to the longitudinal axis of the pier as resolved through the skew angle. The lateral restraint offered by the superstructure is usually ignored. Except in unusual cases, the larger stiffness generated by considering the transverse stiffness of skewed piers is ignored.

13.4.6 Force of Stream Current

The force of flowing water, WA, acting on piers is specified in **LRFD [3.7.3]**. This force acts in both the longitudinal and transverse directions.

13.4.6.1 Longitudinal Force

The longitudinal force is computed as follows:

$$p = \frac{C_D V^2}{1{,}000}$$

Where:

p	=	Pressure of flowing water (ksf)
V	=	Water design velocity for the design flood in strength and service limit states and for the check flood in the extreme event limit state (ft/sec)
C_D	=	Drag coefficient for piers (dimensionless), equal to 0.7 for semicircular-nosed piers, 1.4 for square-ended piers, 1.4 for debris lodged against the pier and 0.8 for wedged-nosed piers with nose angle of 90° or less

The longitudinal drag force shall be computed as the product of the longitudinal stream pressure and the projected exposed pier area.

13.4.6.2 Lateral Force

The lateral force is computed as follows:

$$p = \frac{C_D V^2}{1{,}000}$$

Where:

p	=	Lateral pressure of flowing water (ksf)
C_D	=	Lateral drag coefficient (dimensionless), as presented in Table 13.4-3

Angle Between the Flow Direction and the Pier's Longitudinal Axis	C_D
0°	0.0
5°	0.5
10°	0.7
20°	0.9
≥ 30°	1.0

Table 13.4-3
Lateral Drag Coefficient Values

The lateral drag force shall be computed as the product of lateral stream pressure and the projected exposed pier area. Use the water depth and velocity at flood stage with the force acting at one-half the water depth.

Normally the force of flowing water on piers does not govern the pier design.

13.4.7 Buoyancy

Buoyancy, a component of water load WA, is specified in **LRFD [3.7.2]** and is taken as the sum of the vertical components of buoyancy acting on all submerged components. The footings of piers in the floodplain are to be designed for uplift due to buoyancy.

Full hydrostatic pressure based on the water depth measured from the bottom of the footing is assumed to act on the bottom of the footing. The upward buoyant force equals the volume of concrete below the water surface times the unit weight of water. The effect of buoyancy on column design is usually ignored. Use high water elevation when analyzing the pier for over-turning. Use low water elevation to determine the maximum vertical load on the footing.

The submerged weight of the soil above the footing is used for calculating the vertical load on the footing. Typical values are presented in Table 13.4-4.

	Submerged Unit Weight, γ (pcf)				
	Sand	Sand & Gravel	Silty Clay	Clay	Silt
Minimum (Loose)	50	60	40	30	25
Maximum (Dense)	85	95	85	70	70

Table 13.4-4
Submerged Unit Weights of Various Soils

13.4.8 Ice

Forces from floating ice and expanding ice, IC, do not act on a pier at the same time. Consider each force separately when applying these design loads.

For all ice loads, investigate each site for existing conditions. If no data is available, use the following data as the minimum design criteria:

- Ice pressure = 32 ksf
- Minimum ice thickness = 12”
- Height on pier where force acts is at the 2-year high water elevation. If this value is not available, use the elevation located midway between the high and measured water elevations.
- Pier width is the projection of the pier perpendicular to stream flow.

Slender and flexible piers shall not be used in regions where ice forces are significant, unless approval is obtained from the WisDOT Bureau of Structures.

13.4.8.1 Force of Floating Ice and Drift

Ice forces on piers are caused by moving sheets or flows of ice striking the pier.

There is not an exact method for determining the floating ice force on a pier. The ice crushing strength primarily depends on the temperature and grain size of the ice. **LRFD [3.9.2.1]** sets the effective ice crushing strength at between 8 and 32 ksf.

The horizontal force caused by moving ice shall be taken as specified in **LRFD [3.9.2.2]**, as follows:

$$F = F_c = C_a ptw$$

$$C_a = \left(\frac{5t}{w} + 1 \right)^{0.5}$$

Where:

p	=	Effective ice crushing strength (ksf)
t	=	Ice thickness (ft)
w	=	Pier width at level of ice action (ft)

WisDOT policy item:

Since the angle of inclination of the pier nose with respect to the vertical is always less than or equal to 15° on standard piers in Wisconsin, the flexural ice failure mode does not need to be considered for these standard piers ($f_b = 0$).

WisDOT policy item:

If the pier is approximately aligned with the direction of the ice flow, only the first design case as specified in **LRFD [3.9.2.4]** shall be investigated due to the unknowns associated with the friction angle defined in the second design case.

A longitudinal force equal to F shall be combined with a transverse force of 0.15F

Both the longitudinal and transverse forces act simultaneously at the pier nose.

If the pier is located such that its longitudinal axis is skewed to the direction of the ice flow, the ice force on the pier shall be applied to the projected pier width and resolved into components. In this condition, the transverse force to the longitudinal axis of the pier shall be a minimum of 20% of the total force.

WisDOT exception to AASHTO:

Based upon the pier geometry in the Standards, the ice loadings of **LRFD [3.9.4]** and **LRFD [3.9.5]** shall be ignored.

13.4.8.2 Force Exerted by Expanding Ice Sheet

Expansion of an ice sheet, resulting from a temperature rise after a cold wave, can develop considerable force against abutting structures. This force can result if the sheet is restrained between two adjacent bridge piers or between a bluff type shore and bridge pier. The force direction is therefore transverse to the direction of stream flow.

Force from ice sheets depends upon ice thickness, maximum rate of air-temperature rise, extent of restraint of ice and extent of exposure to solar radiation. In the absence of more precise information, estimate an ice thickness and use a force of 8.0 ksf.

It is not necessary to design all bridge piers for expanding ice. If one side of a pier is exposed to sunlight and the other side is in the shade along with the shore in the pier vicinity, consider the development of pressure from expanding ice. If the central part of the ice is exposed to the sun's radiation, consider the effect of solar energy, which causes the ice to expand.

13.4.9 Centrifugal Force

Centrifugal force, CE, is specified in **LRFD [3.6.3]** and is included in the pier design for structures on horizontal curves. The lane load portion of the HL-93 loading is neglected in the computation of the centrifugal force.

The centrifugal force is taken as the product of the axle weights of the design truck or tandem and the factor, C, given by the following equation:

$$C = \frac{4}{3}\frac{v^2}{gR}$$

Where:

V = Highway design speed (ft/sec)

g = Gravitational acceleration = 32.2 (ft/sec^2)

R = Radius of curvature of travel lane (ft)

The multiple presence factors specified in **LRFD [3.6.1.1.2]** shall apply to centrifugal force.

Centrifugal force is assumed to act radially and horizontally 6' above the roadway surface. The point 6' above the roadway surface is measured from the centerline of roadway. The design speed may be determined from the Wisconsin *Facilities Development Manual*, Chapter 11. It is not necessary to consider the effect of superelevation when centrifugal force is used, because the centrifugal force application point considers superelevation.

13.4.10 Extreme Event Collision Loads

WisDOT policy item:

With regards to **LRFD [3.6.5]** and vehicular collision force, CT, protecting the pier and designing the pier for the 600 kip static force are each equally acceptable. The bridge design engineer should work with the roadway engineer to determine which alternative is preferred.

WisDOT policy item:

Designs for bridge piers adjacent to roadways with a design speed ≤ 40 mph need not consider the provisions of **LRFD [3.6.5]**.

If the design speed of a roadway adjacent to a pier is > 40 mph and the pier is not protected by a TL-5 barrier, embankment or adequate offset, the pier columns/shaft, *only*, shall be strengthened to comply with **LRFD [3.6.5]**. For a multi-column pier the minimum size column shall be 3x4 ft rectangular or 4 ft diameter (consider clearance issues and/or the wide cap required when using 4 ft diameter columns). Solid shaft and hammerhead pier shafts are considered adequately sized.

All multi-columned piers require a minimum of three columns. If a pier cap consists of two or more segments each segment may be supported by two columns. If a pier is constructed in stages, two columns may be used for the temporary condition.

The vertical reinforcement for the columns/shaft shall be the greater of what is required by design (not including the Extreme Event II loading) or a minimum of 1.0% of the gross concrete section (total cross section without deduction for rustications less than or equal to 1-1/2" deep) to address the collision force for the 3x4 ft rectangular and 4 ft diameter columns.

For the 3x4 ft rectangular columns, use double #5 stirrups spaced at 6" vertically as a minimum. For the 4 ft diameter columns, use #4 spiral reinforcement (smooth bars) spaced vertically at 6" as a minimum. Hammerhead pier shafts shall have, as a minimum, the horizontal reinforcement as shown on the Standards.

See Standard for Multi-Columned Pier with Rectangular Columns for an acceptable design to meet **LRFD [3.6.5]**.

WisDOT exception to AASHTO:

The vessel collision load, CV, in **LRFD [3.14]** will not be applied to every navigable waterway of depths greater than 2'. For piers located in navigable waterways, the engineer shall contact the WisDOT project manager to determine if a vessel collision load is applicable.

13.5 Load Application

When determining pier design forces, a thorough understanding of the load paths for each load is critical to arriving at loads that are reasonable per *AASHTO LRFD*. The assumptions associated with different pier, bearing and superstructure configurations are also important to understand. This section provides general guidelines for the application of forces to typical highway bridge piers.

13.5.1 Loading Combinations

Piers are designed for the Strength I, Strength III, Strength V and Extreme Event II load combinations as specified in **LRFD [3.4.1]**. Reinforced concrete pier components are also checked for the Service I load combination. Load factors for these load combinations are presented in Table 13.5-1. See 13.10 for loads applicable to pile bents and pile encased piers.

Load Combination Limit State	Load Factor										
	DC		DW		LL+IM BR CE	WA	WS	WL	FR	TU CR SH	IC CT CV
	Max.	Min.	Max.	Min.							
Strength I	1.25	0.90	1.50	0.65	1.75	1.00	0.00	0.00	1.00	0.50	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.40	0.00	1.00	0.50	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	0.40	1.00	1.00	0.50	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00	0.00
Extreme Event II	1.25	0.90	1.50	0.65	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Table 13.5-1
Load Factors

13.5.2 Expansion Piers

See 13.4 for additional guidance regarding the application of specific loads.

Transverse forces applied to expansion piers from the superstructure include loads from one-half of the adjacent span lengths, and are applied at the location of the transverse load.

For expansion bearings other than elastomeric, longitudinal forces are transmitted to expansion piers through friction in the bearings. These forces, other than temperature, are based on loading one-half of the adjacent span lengths, with the maximum being no greater than the maximum friction force (dead load times the maximum friction coefficient of a sliding bearing). See 27.2.2 to determine the bearing friction coefficient. The longitudinal forces are applied at the bearing elevation.

Expansion piers with elastomeric bearings are designed based on the force that the bearings resist, with longitudinal force being applied at the bearing elevation. This force is applied as some combination of temperature force, braking force, and/or wind load depending on what load case generates the largest deflection at the bearing. The magnitude of the force shall be computed as follows:

$$F = \frac{GA\Delta n}{t}$$

Where:

F	=	Elastomeric bearing force used for pier design (kips)
G	=	Shear modulus of the elastomer (ksi)
A	=	Bearing pad area (in^2)
Δ	=	Deflection at bearing from thermal or braking force (in)
n	=	Number of bearings per girder line; typically one for continuous steel girders and two for prestressed concrete beams (dimensionless)
t	=	Total elastomer thickness (without steel laminates) (in)

Example E27-1.8 illustrates the calculation of this force.

See 13.4.5 for a discussion and example of temperature force application for all piers.

13.5.3 Fixed Piers

Transverse forces applied to expansion piers from the superstructure include loads from one-half of the adjacent span lengths, and are applied at the location of the transverse load. For fixed bearings, longitudinal forces, other than temperature, are based on loading one-half of the adjacent span lengths. The longitudinal forces are applied at the bearing elevation.

See 13.4.5 for a discussion and example of temperature force application for all piers.

13.6 Multi-Column Pier and Cap Design

WisDOT policy item:

Multi-column pier caps shall be designed using conventional beam theory.

The first step in the analysis of a pier frame is to determine the trial geometry of the frame components. The individual components of the frame must meet the minimum dimensions specified in 13.2.1 and as shown on the Standards. Each of the components should be sized for function, economy and aesthetics. Once a trial configuration is determined, analyze the frame and adjust the cap, columns and footings if necessary to accommodate the design loads.

When the length between the outer columns of a pier cap exceeds 65', temperature and shrinkage should be considered in the design of the columns. These effects induce moments in the columns due to the expansion and contraction of the cap combined with the rigid connection between the cap and columns. A 0.5 factor is specified in the strength limit state for the temperature and shrinkage forces to account for the long-term column cracking that occurs. A full section modulus is then used for this multi-column pier analysis. Use an increase in temperature of +35 degrees F and a decrease of -45 degrees F. Shrinkage (0.0003 ft/ft) will offset the increased temperature force. For shrinkage, the keyed vertical construction joint as required on the Standard for Multi-Columned Pier, is to be considered effective in reducing the cap length. For all temperature forces, the entire length from exterior column to exterior column shall be used.

The maximum column spacing on pier frames is 25'. Column height is determined by the bearing elevations, the bottom of footing elevation and the required footing depth. The pier cap/column and column/footing interfaces are assumed to be rigid.

The pier is analyzed as a frame bent by any of the available analysis procedures considering sidesway of the frame due to the applied loading. The gross concrete areas of the components are used to compute their moments of inertia for analysis purposes. The effect of the reinforcing steel on the moment of inertia is neglected.

Vertical loads are applied to the pier through the superstructure. The vertical loads are varied to produce the maximum moments and shears at various positions throughout the structure in combination with the horizontal forces. The effect of length changes in the cap due to temperature is also considered in computing maximum moments and shears. All these forces produce several loading conditions on the structure which must be separated to get the maximum effect at each point in the structure. The maximum moments, shears and axial forces from the analysis routines are used to design the individual pier components. Moments at the face of column are used for pier cap design.

Skin reinforcement on the side of the cap, shall be determined as per **LRFD [5.7.3.4]**. This reinforcement shall not be included in any strength calculations.

See 13.1 and 13.2.1 for further requirements specific to this pier type.

13.7 Hammerhead Pier Cap Design

WisDOT policy item:

Hammerhead pier caps shall be designed using the strut-and-tie method.

The strut-and-tie method is simply the creation of an internal truss system used to transfer the loads from the bearings through the pier cap to the column(s). This is accomplished through a series of concrete "struts" that resist compressive forces and steel "ties" that resist tensile forces. These struts and ties meet at nodes. See Figure 13.7-1 for a basic strut-and-tie model that depicts two bearing reactions transferred to two columns.

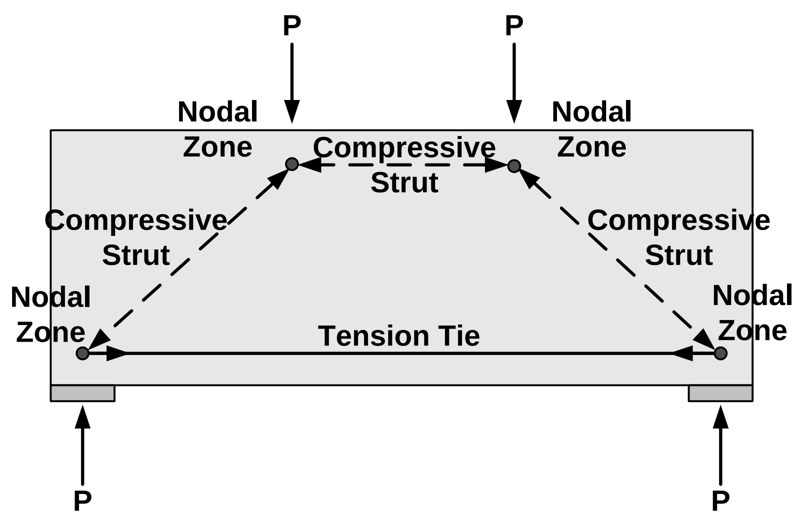


Figure 13.7-1
Basic Strut-and-Tie Elements

Strut-and-tie models are based on the following assumptions:

- The tension ties yield before the compressive struts crush.
- External forces are applied at nodes.
- Forces in the struts and ties are uniaxial.
- Equilibrium is maintained.
- Prestressing of the pier is treated as a load.

The generation of the model requires informed engineering judgment and is an iterative, graphical procedure. The following steps are recommended for a strut-and-tie pier cap design.

13.7.1 Draw the Idealized Truss Model

This model will be based on the structure geometry and loading configuration. At a minimum, nodes shall be placed at each load and support point. Maintain angles of approximately 30° (minimum of 25º) to 60° (maximum of 65º) between truss members. An angle close to 45° should be used when possible. Figure 13.7-2 depicts an example hammerhead pier cap strut-and-tie model.

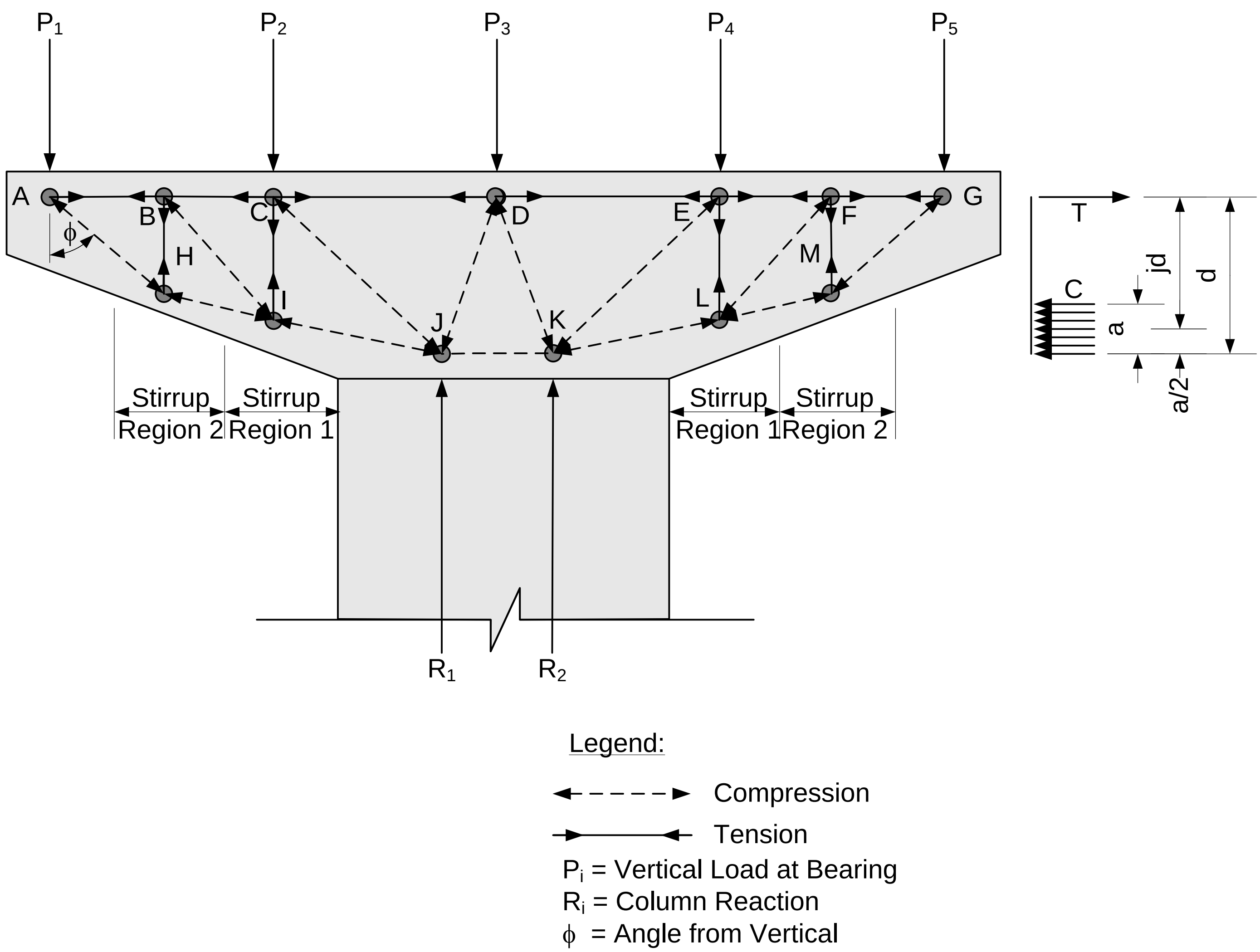


Figure 13.7-2
Example Hammerhead Pier Cap Strut-and-Tie Model

To begin, place nodes at the bearing locations and at the two column 1/3-points. In Figure 13.7-2, the minimum of nodes A, C, D, E and G are all placed at a bearing location, and nodes J and K are placed at the column 1/3-points. When drawing the truss model, the order of priority for forming compressive struts shall be the following:

1. Transfer the load directly to the column if the angle from vertical is less than 60°.

2. Transfer the load to a point directly beneath a bearing if the angle from vertical is between 30° and 60°.

3. Transfer the load at an approximately 45° angle from vertical and form a new node.

In Figure 13.7-2, the bearing load at node C is transferred directly to the column at node J since the angle formed by the compression strut C-J is less than 60°. The same occurs at strut E-K. However, the angle that would be formed by compression strut A-J to the column is not less than 60°, nor is the angle that would be formed by a strut A-I to beneath a bearing. Therefore, the load at node A is transferred at a 45° angle to node H by strut A-H. To maintain equilibrium at node H, the vertical tension tie B-H and the compression strut H-I are added.

Then, since the angle that would be formed by potential column strut B-J is not less than 60°, a check is made of the angle that would be formed by strut B-I. Since this angle is within the 30° to 60° range, compression strut B-I is added. To maintain equilibrium at node I, the vertical tension tie C-I and the compression strut I-J are added. This completes the basic strut-and-tie model for the left side of the cap. The geometric setup on the right side of the cap will be performed in the same manner as the left side.

The bearing load at node D, located above the column, is then distributed directly to the column as the angle from vertical of struts D-J and D-K are both less than 60°. Compression strut J-K must then be added to satisfy equilibrium at nodes J and K.

Vertically, the top chord nodes A, B, C, D, E, F and G shall be placed at the centroid of the tension steel. The bottom chord nodes H, I, J, K, L and M shall follow the taper of the pier cap and be placed at mid-height of the compression block, a/2, as shown in Figure 13.7-2.

The engineer should then make minor adjustments to the model using engineering judgment. In this particular model, this should be done with node H in order to make struts A-H and B-I parallel. The original 45° angle used to form strut A-H likely did not place node H halfway between nodes A and C. The angle of strut A-H should be adjusted so that node H is placed halfway between nodes A and C.

Another adjustment the engineer may want to consider would be placing four nodes above the column at 1/5-points as opposed to the conservative approach of the two column nodes shown in Figure 13.7-2 at 1/3-points. The four nodes would result in a decrease in the magnitude of the force in tension tie C-I. If the structure geometry were such that girder P_2 were placed above the column or the angle from vertical for potential strut B-J were less than 60°, then the tension tie C-I would not be present.

13.7.2 Solve for the Member Forces

Determine the magnitude of the unknown forces in the internal tension ties and compression struts by transferring the known external forces, such as the bearing reactions, through the strut-and-tie model. To satisfy equilibrium, the sum of all vertical and horizontal forces acting at each node must equal zero.

13.7.3 Check the Size of the Bearings

Per **LRFD [5.6.3.5]**, the concrete area supporting the bearing devices shall satisfy the following:

$$P_u \leq \phi P_n$$

Where:

ϕ = Resistance factor for bearing on concrete, equal to 0.70, as specified in **LRFD [5.5.4.2]** (dimensionless)

P_u = Bearing reaction from strength limit state (kips)

P_n = Nominal bearing resistance (kips)

For node regions bounded by compressive struts and bearing areas:

$$P_n = 0.85 f'_c A$$

For node regions anchoring a tension tie in one direction:

$$P_n = 0.75 f'_c A$$

For node regions anchoring tension ties in more than one direction:

$$P_n = 0.65 f'_c A$$

Where:

f'_c = Specified concrete compressive strength (ksi)

A = Area under bearing device (in^2)

WisDOT policy item:

WisDOT standard pier caps satisfy the bearing requirements of **LRFD [5.7.5]**.

13.7.4 Design Tension Tie Reinforcement

Tension ties shall be designed to resist the strength limit state force per **LRFD [5.6.3.4.1]**. For non-prestressed caps, the tension tie steel shall satisfy:

$$A_{st} \geq \frac{P_u}{\phi f_y}$$

Where:

A_{st}	=	Total area of mild steel reinforcement in the tie (in^2)
P_u	=	Tension tie force from strength limit state (kips)
ϕ	=	Resistance factor for tension on reinforced concrete, equal to 0.90, as specified in **LRFD [5.5.4.2]** (dimensionless)
f_y	=	Yield strength of reinforcement (ksi)

Horizontal tension ties, such as ties A-B and E-F in Figure 13.7-2, are used to determine the longitudinal reinforcement required in the top of the pier cap. The maximum tension tie value should be used to calculate the top longitudinal reinforcement.

Vertical tension ties, such as ties B-H and C-I, are used to determine the vertical stirrup requirements in the cap. Similar to traditional shear design, two stirrup legs shall be accounted for when computing A_{st}. In Figure 13.7-2, the number of stirrups, n, necessary to provide the A_{st} required for tie B-H shall be spread out across Stirrup Region 2. The length limits of Stirrup Region 2 are from the midpoint between nodes A and B to the midpoint between nodes B and C. When vertical ties are located adjacent to columns, such as with tie C-I, the stirrup region extends to the column face. Therefore, the length limits of Stirrup Region 1 are from the column face to the midpoint between nodes B and C. The stirrup spacing shall then be determined by the following equation:

$$s_{max} = \frac{L}{n}$$

Where:

s_{max}	=	Maximum allowable stirrup spacing (in)
L	=	Length of stirrup region (in)
n	=	Number of stirrups required to satisfy the A_{st} required to resist the vertical tension tie force

Skin reinforcement on the side of the cap, shall be determined as per **LRFD [5.7.3.4]**. This reinforcement shall not be included in any strength calculations.

13.7.5 Check the Compression Strut Capacity

Compression struts shall be designed to resist the strength limit state force per **LRFD [5.6.3.3]**. The resistance of an unreinforced compression strut shall be taken as:

$$P_r = \phi f_{cu} A_{cs} \geq P_u$$

Where:

P_r	=	Factored resistance of compression strut (kips)
P_u	=	Compression strut force from strength limit state (kips)
ϕ	=	Resistance factor for compression in strut-and-tie models, equal to 0.70, as specified in **LRFD [5.5.4.2]** (dimensionless)
f_{cu}	=	Limiting compressive stress (ksi)
A_{cs}	=	Effective cross-sectional area of strut (in^2)

The limiting compressive stress shall be given by:

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \leq 0.85f'_c$$

In which:

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s$$

Where:

ε_s	=	Concrete tensile strain in the direction of the tension tie at the strength limit state (in/in)
α_s	=	Smallest angle between the compression strut and the adjoining tension ties (°)
f'_c	=	Specified compressive strength (ksi)

The concrete tensile strain is given by:

$$\varepsilon_s = \frac{P_u}{A_{st} E_s}$$

Where:

E_s = Modulus of elasticity of steel, taken as 29,000 (ksi)

The cross-sectional area of the strut, A_{cs}, is determined by considering both the available concrete area and the anchorage conditions at the end of the strut. Figure 13.7-3, Figure 13.7-4 and Figure 13.7-5 illustrate the computation of A_{cs}.

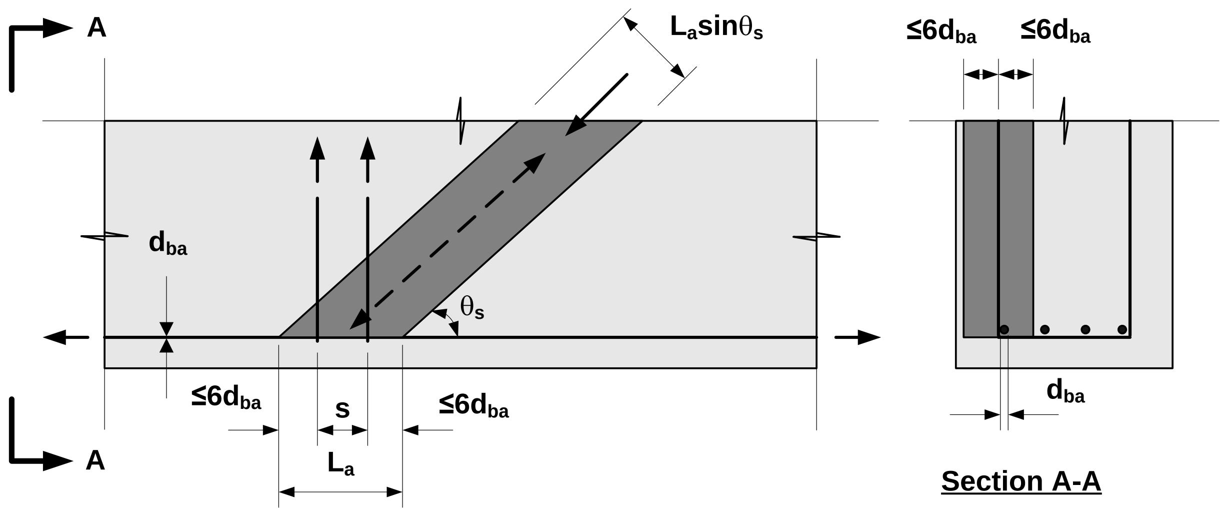


Figure 13.7-3
Strut Anchored by Tension Reinforcement Only

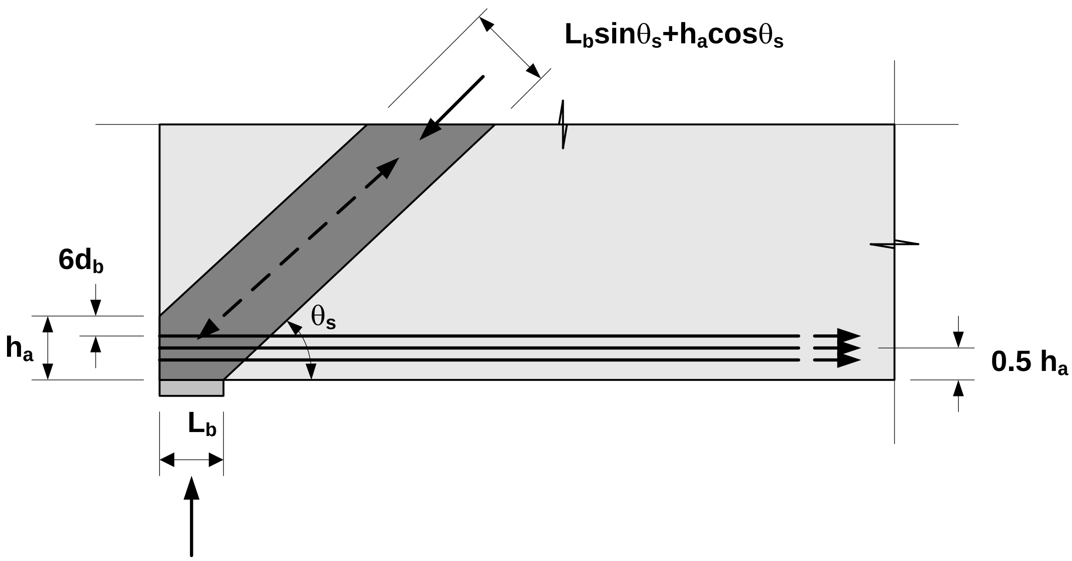


Figure 13.7-4
Strut Anchored by Bearing and Tension Reinforcement

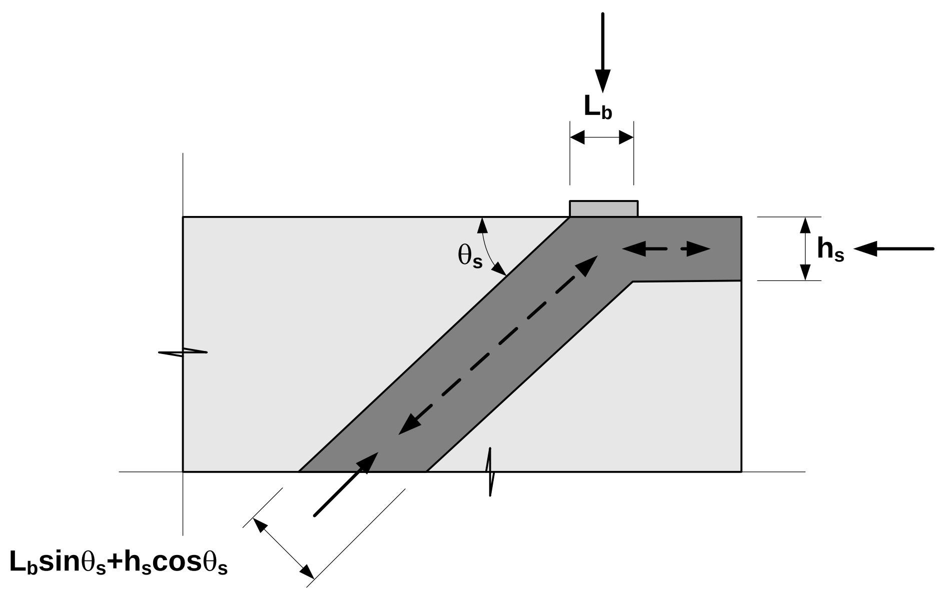


Figure 13.7-5
Strut Anchored by Bearing and Strut

In Figure 13.7-3, the strut area is influenced by the stirrup spacing, s, as well as the diameter of the longitudinal tension steel, d_{ba}. In Figure 13.7-4, the strut area is influenced by the bearing dimensions, L_b, in both directions, as well as the location of the center of gravity of the longitudinal tension steel, $0.5h_a$. In Figure 13.7-5, the strut area is influenced by the bearing dimensions, L_b, in both directions, as well as the height of the compression strut, h_s. The value of h_s shall be taken as equal to “a” as shown in Figure 13.7-2. The strut area in each of the three previous figures depends upon the angle of the strut with respect to the horizontal, θ_s.

If the initial strut width is inadequate to develop the required resistance, the engineer should increase the bearing block size.

13.7.6 Check the Tension Tie Anchorage

Tension ties shall be anchored to the nodal zones by either specified embedment length or hooks so that the tension force may be transferred to the nodal zone. As specified in **LRFD [5.6.3.4]**, the tie reinforcement shall be fully developed at the inner face of the nodal zone. In Figure 13.7-4, this location is given by the edge of the bearing where θ_s is shown.

13.7.7 Provide Crack Control Reinforcement

Pier caps designed using the strut-and-tie method shall contain an orthogonal grid of reinforcing bars near each face in accordance with **LRFD [5.6.3.6]**. This reinforcement will control crack widths and ensure a minimum ductility. The ratio of reinforcement area to gross concrete area shall not be less than 0.003 in both directions. Maximum bar spacing shall not exceed 12”. The crack control steel, when located within the tension tie, may be considered as part of the tension tie reinforcement.

13.8 General Pier Cap Information

The minimum cap dimension to be used is 3' deep by 2'-6" wide, with the exception that a 2'-6" deep section may be used for caps under slab structures. If a larger cap is needed, use 6" increments to increase the size. The multi-column cap width shall be a minimum of 1 1/2" wider than the column on each side to facilitate construction forming. The pier cap length shall extend a minimum of 2' transversely beyond the centerline of bearing and centerline of girder intersection.

On continuous slab structures, the moment and shear forces are proportional between the transverse slab section and the cap by the ratio of their moments of inertia. The effective slab width assumed for the transverse beam is the minimum of 1/2 the center-to-center column spacing or 8.0'.

$$M_{cap} = M_{total} \frac{I_{cap}}{I_{cap} + I_{slab}}$$

Where:

M_{cap}	=	Cap moment (kip-ft)
M_{total}	=	Total moment (kip-ft)
I_{cap}	=	Moment of inertia of pier cap (in^4)
I_{slab}	=	Moment of inertia of slab (in^4)

The concrete slab is to extend beyond the edge of pier cap as shown on Standards for Continuous Haunched Slab and for Continuous Flat Slab. If the cap is rounded, measure from a line tangent to the pier cap end and parallel to the edge of the deck.

Reinforcement bars are placed straight in the pier cap. Determine bar cutoff points on wide caps. If the pier cap is cantilevered over exterior columns, the top negative bar steel may be bent down at the ends to ensure development of this primary reinforcement.

Do not place shear stirrups closer than 4" on centers. Generally only double stirrups are used, but triple stirrups may be used to increase the spacing. If these methods do not work, increase the cap size. Stirrups are generally not placed over the columns. The first stirrup is placed one-half of the stirrup spacing from the edge of the column into the span.

The cap-to-column connection is made by extending the column reinforcement straight into the cap the necessary development length. Stirrup details and bar details at the end of the cap are shown on Standard for Multi-Columned Pier.

Crack control, as defined in **LRFD [5.7.3.4]** shall be considered for pier caps. Class 2 exposure condition exposure factors shall only be used when concern regarding corrosion (i.e., pier caps located below expansion joints, pier caps subject to intermittent moisture above waterways, etc.) or significant aesthetic appearance of the pier cap is present.

13.9 Column / Shaft Design

See 13.4.10 for minimum shaft design requirements regarding the Extreme Event II collision load of **LRFD [3.6.5]**.

Use an accepted analysis procedure to determine the axial load as well as the longitudinal and transverse moments acting on the column. These forces are generally largest at the top and bottom of the column. Apply the load factors for each applicable limit state. The load factors should correspond to the gross moment of inertia. Load factors vary for the gross moment of inertia versus the cracked moment as defined in **LRFD [3.4.1]** for γ_{TU}, γ_{CR}, γ_{SH}. Choose the controlling load combinations for the column design.

Columns that are part of a pier frame have transverse moments induced by frame action from vertical loads, wind loads on the superstructure and substructure, wind loads on live load, thermal forces and centrifugal forces. If applicable, the load combination for Extreme Event II must be considered. Longitudinal moments are produced by the above forces, as well as the braking force. These forces are resolved through the skew angle of the pier to act transversely and longitudinally to the pier frame. Longitudinal forces are divided equally among the columns.

Wisconsin uses tied columns following the procedures of **LRFD [5.7.4]**. The minimum allowable column size is 2'-6" in diameter. The minimum steel bar area is as specified in **LRFD [5.7.4.2]**. For piers not requiring a certain percentage of reinforcement as per 13.4.10 to satisfy **LRFD [3.6.5]** for vehicular collision load, a reduced effective area of reinforcement may be used when the cross-section is larger than that required to resist the applied loading.

The computed column moments are to consider moment magnification factors for slenderness effects as specified in **LRFD [5.7.4.3]**. Values for the effective length factor, K, are as follows:

- 1.2 for longitudinal moments with a fixed seat supporting prestressed concrete girders
- 2.1 for longitudinal moments with a fixed seat supporting steel girders and all expansion bearings
- 1.0 for all transverse moments

The computed moments are multiplied by the moment magnification factors, if applicable, and the column is designed for the combined effects of axial load and bending. According to **LRFD [5.7.4.1]** all force effects, including magnified moments, shall be transferred to adjacent components. The design resistance under combined axial load and bending is based on stress-strain compatibility. A computer program is recommended for determining the column's resistance to the limit state loads.

As a minimum, the column shall provide the steel shown on the Pier Standards.

On large river crossings, it may be necessary to protect the piers from damage. Dolphins may be provided.

The column-to-cap connection is designed as a rigid joint considering axial and bending stresses. Column steel is run through the joint into the cap to develop the compressive stresses or the tensile steel stresses at the joint.

In general, the column-to-footing connection is also designed as a rigid joint. The bar steel from the column is generally terminated at the top of the footing. Dowel bars placed in the footing are used to transfer the steel stress between the footing and the column.

Crack control, as defined in **LRFD [5.7.3.4]** shall be considered for pier columns. All pier columns shall be designed using a Class 2 exposure condition exposure factor.

13.10 Pile Bent and Pile Encased Pier Analysis

WisDOT policy item:

Only the Strength I limit state need be utilized for determining the pile configuration required for open pile bents and pile encased piers. Longitudinal forces are not considered due to fixed or semi-expansion abutments being required for these pier types.

The distribution of dead load to the pile bents and pile encased piers is in accordance with 17.2.9. Live load is distributed according to 17.2.10.

WisDOT policy item:

Dynamic load allowance, IM, is included for determining the pile loads in pile bents, but not for piling in pile encased piers.

The pile force in the outermost, controlling pile is equal to:

$$P_n = \frac{F}{n} + \frac{M}{S}$$

Where:

F = Total factored vertical load (kips)

n = Number of piles

M = Total factored moment about pile group centroid (kip-ft)

S = Section modulus of pile group (ft^3), equal to:

$$\left(\frac{\sum d^2}{c}\right)$$

In which:

d = Distance of pile from pile group centroid

c = Distance from outermost pile to pile group centroid

See Standard for Pile Bent for details. See Standard for Pile Encased Pier for details.

13.11 Footing Design

13.11.1 General Footing Considerations

There are typical concepts to consider when designing and detailing both spread footings and pile footings.

For multi-columned piers:

- Each footing for a given pier should be the same dimension along the length of the bridge.
- Each footing for a given pier should be the same thickness.
- Footings within a given pier need not be the same width.
- Footings within a given pier may have variable reinforcement.
- Footings within a given pier may have a different number of piles. Exterior footings should only have fewer piles than an interior footing if the bridge is unlikely to be widened in the future. An appropriate cap span layout will usually lend itself to similar footing/pile configurations.
- Heavier piles, especially if primarily end bearing piles, can be more economical.

For hammerhead piers:

- Make as many seals the same size as reasonable to facilitate cofferdam re-use.
- Seal thickness can vary from pier to pier.
- Footing dimensions, reinforcement and pile configuration can vary from pier to pier.
- Heavier piles, especially if primarily end bearing piles, can be more economical.

WisDOT exception to AASHTO:

Crack control, as defined in **LRFD [5.7.3.4]** shall not be considered for pier isolated spread footings, isolated pile footings and continuous footings.

Shrinkage and temperature reinforcement, as defined in **LRFD [5.10.8]** shall not be considered for side faces of any buried footings.

13.11.2 Isolated Spread Footings

WisDOT policy item:

Spread footings are designed using LRFD strength limit state loads and resistance for moment and shear as specified in **LRFD [5.13.3]**. The foundation bearing capacity, used to dimension the footing's length and width, shall be determined using 4th Edition of the AASHTO *LRFD Bridge Design Specifications for Highway Bridges*.

The spread footing is proportioned so that the foundation bearing capacity is not exceeded. The following steps are used to design spread footings:

1. Minimum depth of spread footings is 2'. Depth is generally determined from shear strength requirements. Shear reinforcement is not used.

2. A maximum of 25% of the footing area is allowed to act in uplift (or nonbearing). When part of a footing is in uplift, its section properties for analysis are based only on the portion of the footing that is in compression (or bearing). When determining the percent of a footing in uplift, use the Service Load Design method.

3. Soil weight on footings is based only on the soil directly above the footing.

4. The minimum depth for frost protection from top of ground to bottom of footing is 4'.

5. Spread footings on seals are designed by either of the following methods:

    a. The footing is proportioned so the pressure between the bottom of the footing and the top of the seal does not exceed the foundation bearing capacity and not more than 25% of the footing area is in uplift.

    b. The seal is proportioned so that pressure at the bottom of the seal does not exceed the foundation bearing capacity and the area in uplift between the footing and the seal does not exceed 25%.

6. The spread footing's reinforcing steel is determined from the flexural requirements of **LRFD [5.7.3]**. The design moment is determined from the volume of the pressure diagram under the footing which acts outside of the section being considered. The weight of the footing and the soil above the footing is used to reduce the bending moment.

7. The negative moment which results if a portion of the footing area is in uplift is ignored. No negative reinforcing steel is used in spread footings.

8. Shear resistance is determined by the following two methods:

    a. Two-way action

The volume of the pressure diagram on the footing area outside the critical perimeter lines (placed at a distance d/2 from the face of the column, where d equals the effective footing depth) determines the shear force. The shear resistance is influenced by the concrete strength, the footing depth and the critical perimeter length. The critical perimeter length is 2 (L + d + W + d) for rectangular columns and π (2R + d) for round columns, where R is the column radius and d is the effective footing depth. The critical perimeter location for spread footings with rectangular columns is illustrated in Figure 13.11-1.

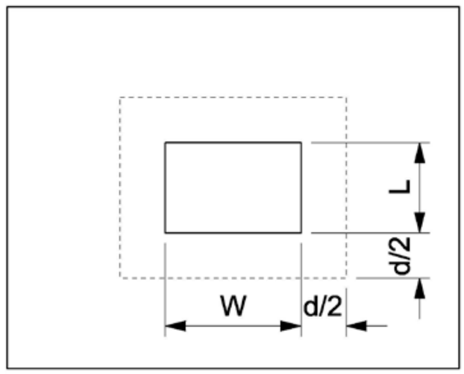


Figure 13.11-1
Critical Perimeter Location for Spread Footings

b. One-way action

The volume of the pressure diagram on the area enclosed by the footing edges and a line placed at a distance "d" from the face of the column determines the shear force. The shear resistance is influenced by the concrete strength, the footing depth and the entire footing width or length. The shear location for one-way action is illustrated in Figure 13.11-2.

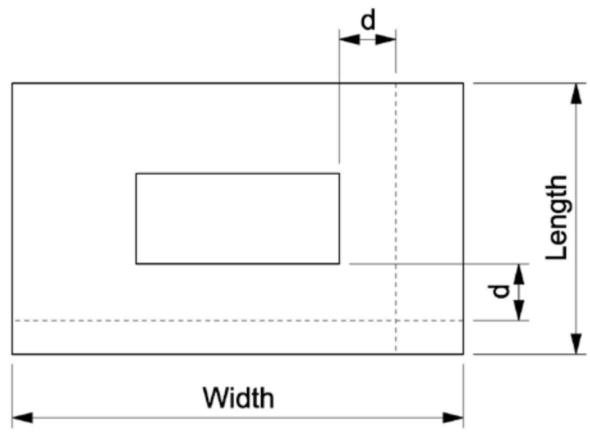


Figure 13.11-2
Shear Location for One-Way Action

The footing weight and the soil above the areas are used to reduce the shear force.

9. The bottom layer of reinforcing steel is placed 3” clear from the bottom of the footing.

10. If adjacent edges of isolated footings are closer than 4’-6”, a continuous footing shall be used.

13.11.3 Isolated Pile Footings

WisDOT policy item:

Pile footings are designed using LRFD strength limit state loads and resistance for moment and shear as specified in **LRFD [5.13.3]**. The pile design shall use LRFD strength limit state loads to compare to the factored axial compression resistance specified in Table 11.3-5.

The nominal geotechnical pile resistance shall be provided in the Site Investigation Report. The engineer shall then apply the appropriate resistance factor from Table 11.3.1 to the nominal resistance to determine the factored pile resistance. The footing is proportioned so that when it is loaded with the strength limit state loads, the factored pile resistance is not exceeded.

The following steps are used to design pile-supported footings:

1. The minimum depth of pile footing is 2’-6”. The minimum pile embedment is 6”. See 13.2.2 for additional information about pile footings used for pile bents.

2. Pile footings in uplift are usually designed by method (a) stated below. However, method (b) may be used if there is a substantial cost reduction.

a. Over one-half of the piles in the footings must be in compression for the Strength limit states. The section properties used in analysis are based only on the piles in compression. Pile and footing (pile cap) design is based on the Strength limit states. Service limit states require check for overall stability; however a check of crack control is not required per 13.11. The 600 kip collision load need not be checked per 13.4.10.

b. Piles may be designed for upward forces provided an anchorage device, sufficient to transfer the load, is provided at the top of the pile. Provide reinforcing steel to resist the tension stresses at the top of the footing.

3. Same as spread footing.

4. Same as spread footing.

5. The minimum number of piles per footing is four.

6. Pile footings on seals are analyzed above the seal. The only effect of the seal is to reduce the pile resistance above the seal by the portion of the seal weight carried by each pile.

7. If no seal is required but a cofferdam is required, design the piles to use the minimum required batter. This reduces the cofferdam size necessary to clear the battered piles since all piles extend above water to the pile driver during driving.

8. The pile footing reinforcing steel is determined from the flexural requirements of **LRFD [5.7.3]**. The design moment and shear are determined from the force of the piles which act outside of the section being considered. The weight of the footing and the soil above the footing are used to reduce the magnitude of the bending moment and shear force.

9. Shear resistance is determined by the following two methods:

a. Two-way action

The summation of the pile forces outside the critical perimeter lines placed at a distance d/2 from the face of the column (where d equals the effective footing depth) determines the shear force. The shear resistance is influenced by the concrete strength, the footing depth and the critical perimeter length. The critical perimeter length is 2 (L + d + W + d) for rectangular columns and π(2R + d) for round columns. The critical perimeter location for pile footings with rectangular columns is illustrated in Figure 13.11-3.

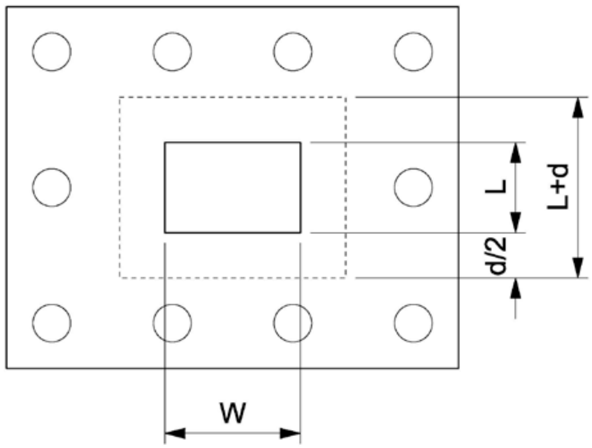


Figure 13.11-3
Critical Perimeter Location for Pile Footings

If the center of a pile falls on a line, then one-half of the pile force is assumed to act on each side of the line.

b. One-way action

The summation of the pile forces located within the area enclosed by the footing edges and a line at distance "d" from the face of the column determines the shear force, as illustrated in Figure 13.11-2. The shear resistance is influenced by the concrete strength, the footing depth and the entire footing width or length. If the center of a pile falls on a line, then one-half of the pile force is assumed to act on each side of the line.

10. The weight of the footing and soil above the areas is used to reduce the shear force.

11. The bottom layer of reinforcing steel is placed directly on top of the piles.

13.11.4 Continuous Footings

Continuous footings are used in pier frames of two or more columns when the use of isolated footings would result in a distance of less than 4’-6” between edges of adjacent footings. They are designed for the moments and shears produced by the frame action of the pier and the soil pressure under the footing.

The soil pressure or pile load under the footing is assumed to be uniform. The soil pressures or pile loads are generally computed only from the vertical column loads along with the soil and footing dead load. The moments at the base of the column are ignored for soil or pile loads.

To prevent unequal settlement, proportion the continuous footing so that soil pressures or pile loads are constant for Service I load combination. The footing should be kept relatively stiff between columns to prevent upward footing deflections which cause excessive soil or pile loads under the columns.

13.11.5 Cofferdams and Seals

A cofferdam is a temporary structure used to construct concrete substructures in or near water. The cofferdam protects the substructure during construction, controls sediments, and can be dewatered to construct the substructure in a dry environment. Dewatering the cofferdam allows for the cutting of piles, placement of reinforcing steel and ensuring proper consolidation of concrete. A cofferdam typically consists of driven steel sheet piling and allows for the structure to be safely dewatered when properly designed. Alternative cofferdam systems may be used to control shallow water conditions.

A cofferdam bid item may be warranted when water is expected at a concrete substructure unit during construction. The cofferdam shall be practically watertight to allow for dewatering such that the substructure is constructed in a dry environment. An exception is for pile encased piers with expected water depths of 5 feet or less. These substructures may be poured underwater, but in certain cases may still require a cofferdam for protection and/or to address environmental concerns. A pile encased pier with expected water depths greater than 5 feet will typically require a cofferdam. The designer should consult with geotechnical and regional personnel to determine if a cofferdam is required. If a cofferdam is warranted, then include the bid item “Cofferdams (Structure)”.

Environmental concerns (specifically sediment control) may require the use of cofferdams at some sites. When excavation takes place in the water, some form of sediment control is usually required. The use of simple turbidity barrier may not be adequate based on several considerations (water depth, velocity, soil conditions, channel width, etc.). All sediment control devices, such as turbidity barrier, shall not be included in structure plans. Refer to Chapter 10 of the FDM for erosion control and storm water quality information.

A seal is a mat of unreinforced concrete poured under water inside a cofferdam. The seal is designed to withstand the hydrostatic pressure on its bottom when the water above it is removed. For shallow water depths and certain soil conditions a concrete seal may not be necessary in order to dewater a cofferdam. Coordinate with geotechnical personnel to determine if a concrete seal is required. The designer shall determine if a concrete seal is required for a cofferdam. If a concrete seal is required, then include the bid item “Concrete Masonry Seal” and required seal dimensions. The cofferdam design shall be the responsibility of the contractor.

The hydrostatic pressure under the seal is resisted by the seal weight, the friction between the seal perimeter and the cofferdam walls, and friction between seal and piles for pile footings. The friction values used for the seal design are considered using the service limit state. To compute the capacity of piles in uplift, refer to Chapter 11. Values for bond on piles and sheet piling are presented in Table 13.11-1.

Application	Value of Bond
Bond on Piles	10 psi
Bond on Sheet Piling	2 psi applied to [(Seal Depth - 2') x Seal Perimeter]

Table 13.11-1
Bond on Piles and Sheet Piling

Lateral forces from stream flow pressure are resisted by the penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. When seals for spread footings are founded on rock, the weight of the seal is used to counterbalance the lateral stream flow pressure.

The downstream side of the cofferdam should be keyed into rock deep enough or other measures should be used to resist the lateral stream flow pressure. To provide a factor of safety, the cofferdam weight (sheet piling and bracing) is ignored in the analysis. The design stream flow velocity is based on the flow at the site at the time of construction but need not exceed 75% of the 100-year velocity. The force is calculated as per 13.4.6.

A rule of thumb for seal thickness is 0.40H for spread footings and 0.25H for pile footings, where H is the water depth from bottom of seal to top of water. The 2-year high water elevation, if available, should be used as the estimated water elevation during construction. The assumed water elevation used to determine the seal thickness should be noted on the plans. The minimum seal size is 3'-0" larger than the footing size on all sides. See Standard for Hammerhead Pier for additional guidance regarding the sizing of the seal.

Example: Determine the seal thickness for a 9' x 12' footing with 12-12" diameter piles. Uplift capacity of one pile equals 15 kips (per the Geotechnical Engineer). The water depth to the top of seal is 16'.

Assume 15' x 18' x 3.25' seal.

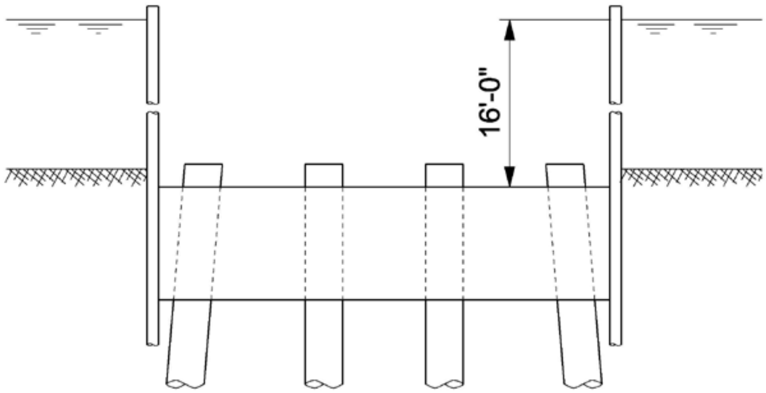


Figure 13.11-4
Seal Inside a Cofferdam

Uplift force of water	15 x 18 x 19.25 x 0.0624	=	324.3 kips (up)
Weight of seal course	15 x 18 x 3.25 x 0.15	=	131.6 kips (down)
Friction of sheet piling	2 x (15+18) x (3.25 - 2.0) x 144 x 0.002	=	23.8 kips (down)
Pile frictional resistance	π x 12 x (3.25 x 12) x 0.010	=	14.7 kips
Pile uplift resistance	(Per Geotechnical Engineer)	=	15.0 kips
Total pile resistance	12 piles x min(14.7,15.0)	=	176.4 kips (down)
Sum of downward forces	131.6+23.8+176.4	=	332 kips
Sum of upward forces	324.3	=	324 kips

332 > 324 OK

USE 3'- 3" THICK SEAL

Note: Pile uplift resistance shall be determine by the Geotechnical Engineer. For this example, when the pile uplift resistance equals 10 kips a 4'-6" thick seal is required.

13.12 Quantities

Consider the "Upper Limits for Excavation" for piers at such a time when the quantity is a minimum. This is either at the existing ground line or the finished graded section. Indicate in the general notes which value is used.

Structure backfill is not used at piers except under special conditions.

Compute the concrete quantities for the footings, columns and cap, and show values for each of them on the final plans.

13.13 Design Examples

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E13-1 Hammerhead Pier Design Example

This example shows design calculations conforming to the ***AASHTO LRFD Bridge Design Specifications (Seventh Edition - 2015 Interim)*** as supplemented by the *WisDOT Bridge Manual* The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-1.1 Obtain Design Criteria

This pier is designed for the superstructure as detailed in example **E24-1**. This is a two-span steel girder stream crossing structure. Expansion bearings are located at the abutments, and fixed bearings are used at the pier.

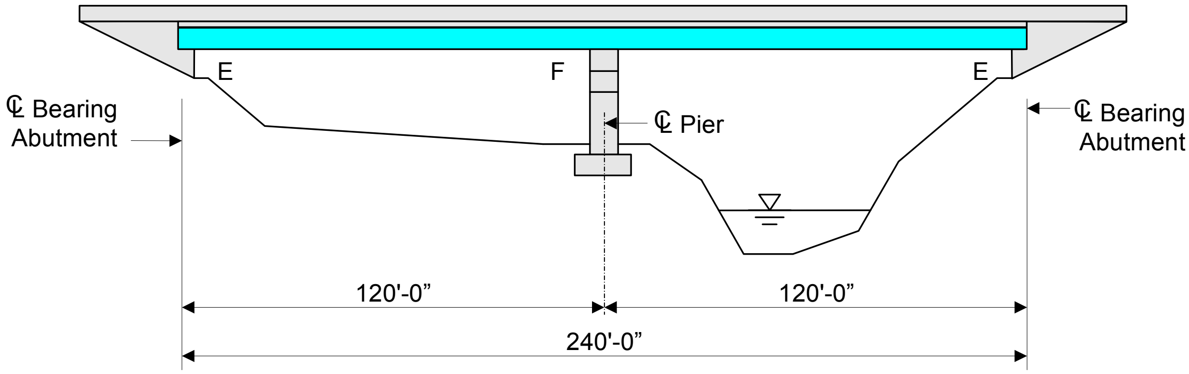

Figure E13-1.1-1
Bridge Elevation

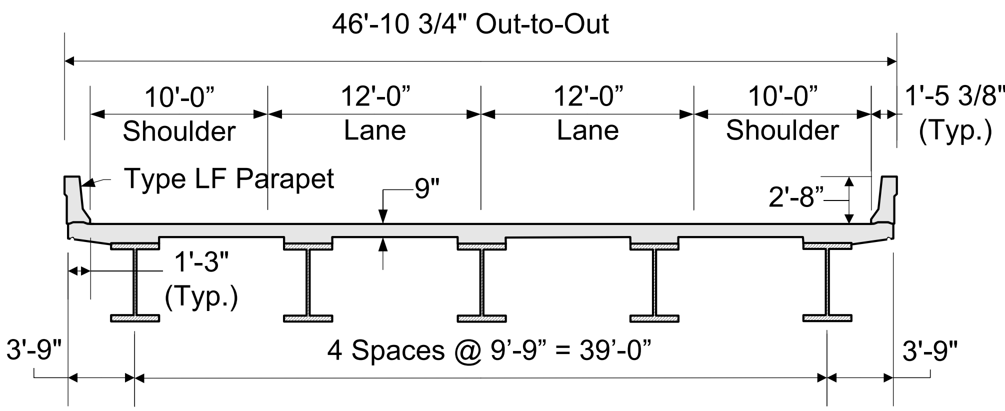

Figure E13-1.1-2
Bridge Cross Section

E13-1.1.1 Material Properties:

$w_c := 0.150$	unit weight of concrete, kcf
$f'_c := 3.5$	concrete 28-day compressive strength, ksi
$f_y := 60$	reinforcement strength, ksi

E13-1.1.2 Reinforcing steel cover requirements:

All cover dimensions listed below are in accordance with **LRFD [Table 5.12.3-1]** and are showr in inches.

$Cover_{cp} := 2.5$	Pier cap
$Cover_{co} := 2.5$	Pier column
$Cover_{ft} := 2.0$	Footing top cover
$Cover_{fb} := 6.0$	Footing bottom cover, based on standard pile projection

E13-1.2 Relevant superstructure data

$w_{deck} := 46.50$	Deck Width, ft
$w_{roadway} := 44.0$	Roadway Width, ft
$ng := 5$	Number of Girders
$S := 9.75$	Girder Spacing, ft
$DOH := 3.75$	Deck Overhang, ft (Note that this overhang exceeds the limits stated in Chapter 17.6.2. WisDOT practice is to limit the overhang to 3'-7".)
$N_{spans} := 2$	
$L := 120.0$	Span Length, ft
$skew := 0$	Degrees
$H_{super} = 8.46$	Superstructure Depth, ft
$H_{brng} := 6.375$	Bearing Height, in (Fixed, Type A)
$W_{brng} := 18$	Bearing Width, in
$L_{brng} := 26$	Bearing Length, in
$\mu_{max} := 0.10$	Max. Coefficient of Friction of Abutment Expansion Bearings
$\mu_{min} := 0.06$	Min. Coefficient of Friction of Abutment Expansion Bearings

E13-1.2.1 Girder Dead Load Reactions

Unfactored Dead Load Reactions, kips

$$DLR_{int} := \begin{pmatrix} \text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\ \text{"Beam"} & 7.00 & 34.02 \\ \text{"Misc"} & 1.23 & 4.73 \\ \text{"Deck"} & 46.89 & 178.91 \\ \text{"Parapet"} & 6.57 & 24.06 \\ \text{"FWS"} & 7.46 & 27.32 \end{pmatrix} \qquad DLR_{ext} := \begin{pmatrix} \text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\ \text{"Beam"} & 7.00 & 34.02 \\ \text{"Misc"} & 0.83 & 3.15 \\ \text{"Deck"} & 48.57 & 185.42 \\ \text{"Parapet"} & 6.57 & 24.06 \\ \text{"FWS"} & 7.46 & 27.32 \end{pmatrix}$$

$\boxed{AbutRint_{DC} = 61.69}$ kips $\boxed{AbutRext_{DC} = 62.97}$ kips

$\boxed{AbutRint_{DW} = 7.46}$ kips $\boxed{AbutRext_{DW} = 7.46}$ kips

Pier Reactions:

$\boxed{Rint_{DC} = 241.72}$ kips $\boxed{Rext_{DC} = 246.65}$ kips

$\boxed{Rint_{DW} = 27.32}$ kips $\boxed{Rext_{DW} = 27.32}$ kips

E13-1.2.2 Live Load Reactions per Design Lane

Unfactored Live Load Reactions, kips

$$LLR := \begin{pmatrix} \text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\ \text{"Vehicle"} & 64.72 & 114.17 \\ \text{"Lane"} & 32.76 & 89.41 \end{pmatrix}$$

These loads are per design lane and do not include dynamic load allowance. The pier reactions are controlled by the 90% (Truck Pair + Lane) loading condition. The reactions shown include the 90% factor.

E13-1.3 Select Preliminary Pier Dimensions

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. For this design example, a single column (hammerhead) pier was chosen.

Since the Specifications do not have standards regarding maximum or minimum dimensions for a pier cap, column, or footing, the designer should base the preliminary pier dimensions on WisDOT specific standards, previous designs, and past experience. The pier cap, however, must be wide enough to accommodate the bearing.

Figures E13-1.3-1 and E13-1.3-2 show the preliminary dimensions selected for this pier design example.

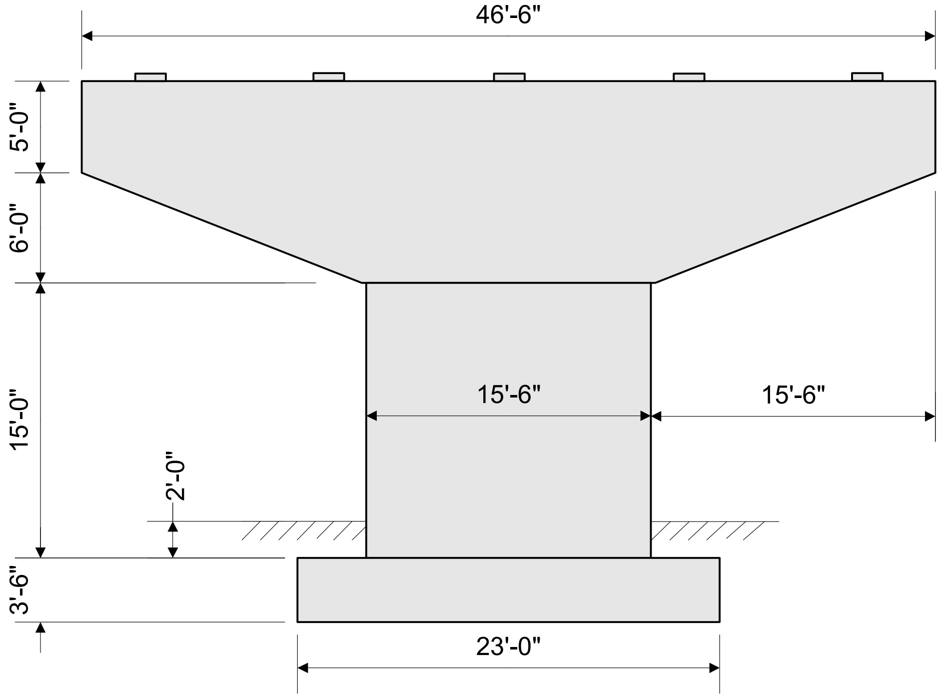

Figure E13-1.3-1
Preliminary Pier Dimensions - Front Elevation

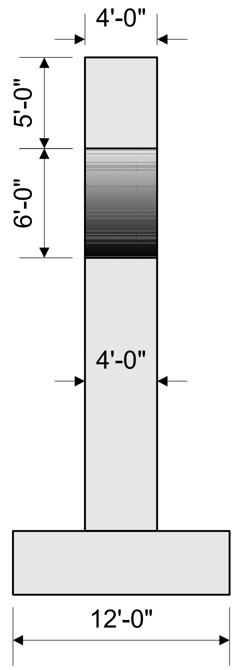

Figure E13-1.3-2
Preliminary Pier Dimensions - End Elevation

Pier Geometry Definitions (feet):

$L_{cap} := 46.5$ $L_{col} := 15.5$ $L_{ftg} := 23$ $D_{soil} := 2$

$W_{cap} := 4$ $W_{col} := 4$ $W_{ftg} := 12$ $\gamma_{soil} := 0.120$

$H_{cap} := 11$ $H_{col} := 15$ $H_{ftg} := 3.5$

$H_{cap\_end} := 5$

$L_{oh} := 15.5$

E13-1.4 Compute Dead Load Effects

Once the preliminary pier dimensions are selected, the corresponding dead loads can be computed in accordance with **LRFD [3.5.1]**. The pier dead loads must then be combined with the superstructure dead loads.

Exterior girder dead load reactions (DC and DW): $\boxed{Rext_{DC} = 246.65}$ kips

$\boxed{Rext_{DW} = 27.32}$ kips

Interior girder dead load reactions (DC and DW): $\boxed{Rint_{DC} = 241.72}$ kips

$\boxed{Rint_{DW} = 27.32}$ kips

Pier cap dead load:

$$DL_{Cap} := w_c \cdot W_{cap} \cdot \left[2 \cdot \left(\frac{H_{cap\_end} + H_{cap}}{2}\right) \cdot L_{oh} + H_{cap} \cdot L_{col}\right]$$

$$= 0.150 \cdot 4 \cdot \left(2 \cdot \frac{5 + 11}{2} \cdot 15.5 + 11 \cdot 15.5\right) \qquad \boxed{DL_{Cap} = 251.1} \text{ kips}$$

Pier column dead load:

$$DL_{col} := w_c \cdot W_{col} \cdot H_{col} \cdot L_{col}$$

$$= 0.150 \cdot 4 \cdot 15 \cdot 15.5 \qquad \boxed{DL_{col} = 139.5} \text{ kips}$$

Pier footing dead load:

$$DL_{ftg} := w_c \cdot W_{ftg} \cdot H_{ftg} \cdot L_{ftg}$$

$$= 0.150 \cdot 12 \cdot 3.5 \cdot 23 \qquad \boxed{DL_{ftg} = 144.9} \text{ kips}$$

In addition to the above dead loads, the weight of the soil on top of the footing must be computed. The two-foot height of soil above the footing was previously defined. Assuming a unit weight of soil at 0.120 kcf in accordance with **LRFD [Table 3.5.1-1]** :

$$EV_{ftg} := \gamma_{soil} \cdot D_{soil} \cdot \left(W_{ftg} \cdot L_{ftg} - W_{col} \cdot L_{col}\right)$$

$$= 0.120 \cdot 2 \cdot (12 \cdot 23 - 4 \cdot 15.5) \qquad \boxed{EV_{ftg} = 51.36} \text{ kips}$$

E13-1.5 Compute Live Load Effects

For the pier in this design example, the maximum live load effects in the pier cap, column and footing are based on either one, two or three lanes loaded (whichever results in the worst force effect). Figure E13-1.5-1 illustrates the lane positions when three lanes are loaded.

The positioning shown in Figure E13-1.5-1 is determined in accordance with **LRFD [3.6.1].** The first step is to calculate the number of design lanes, which is the integer part of the ratio of the clear roadway width divided by 12 feet per lane. Then the lane loading, which occupies ten feet of the lane, and the HL-93 truck loading, which has a six-foot wheel spacing and a two-foot clearance to the edge of the lane, are positioned within each lane to maximize the force effects in each of the respective pier components.

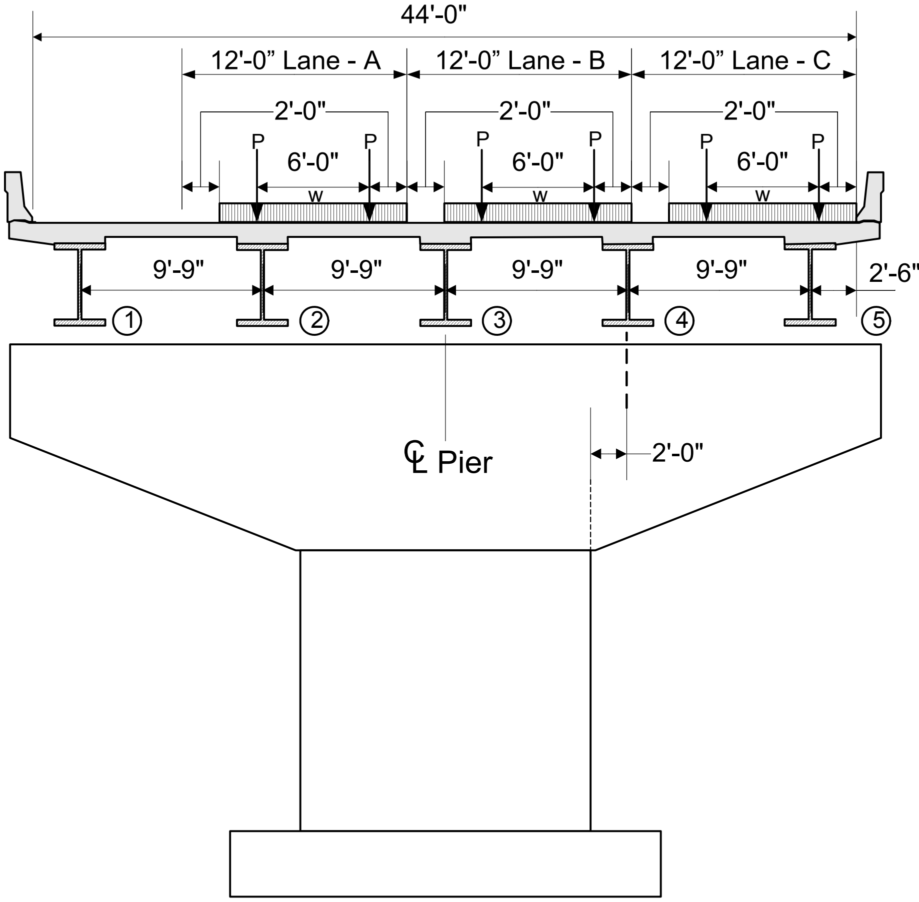

Figure E13-1.5-1
Pier Live Loading

N = maximum number of design lanes that the bridge can accommodate

$w_{roadway}$ = roadway width between curbs, ignoring any median strip

W = design lane width

$W := 12$ feet

$\boxed{w_{roadway} = 44}$ feet

$$N := \frac{w_{roadway}}{W}$$

$\boxed{N = 3.67}$

$\boxed{N = 3}$ design lanes

The unfactored girder reactions for lane load and truck load are obtained from the superstructure analysis and are as shown in E13-1.1.3.2. These reactions do not include dynamic load allowance and are given on a per lane basis (i.e., distribution factor = 1.0). Also, the reactions include the ten percent reduction permitted by the Specifications for interior pier reactions that result from longitudinally loading the superstructure with a truck pair in conjunction with lane loading **LRFD [3.6.1.3.1]**.

$\boxed{R_{truck} = 114.17}$ kips

$\boxed{R_{lane} = 89.41}$ kips

$IM := 0.33$ Dynamic load allowance, IM from **LRFD [Table 3.6.2.1-1]**

The values of the unfactored concentrated loads which represent the girder truck pair load reaction per wheel line in Figure E13-1.5-1 are:

$$P_{wheel} := \frac{R_{truck}}{2} \cdot (1 + IM) \qquad \boxed{P_{wheel} = 75.92} \text{ kips}$$

The value of the unfactored uniformly distributed load which represents the girder lane load reaction in Figure E13-1.5-1 is computed next. This load is transversely distributed over ten feet and is not subject to dynamic load allowance, **LRFD [3.6.2.1]**.

$$W_{lane} := \frac{R_{lane}}{10} \qquad \boxed{W_{lane} = 8.94} \ \frac{\text{kips}}{\text{ft}}$$

The next step is to compute the reactions due to the above loads at each of the five bearing locations. This is generally carried out by assuming the deck is pinned (i.e., discontinuous) at the interior girder locations but continuous over the exterior girders. Solving for the reactions is then elementary. The computations for the reactions with only Lane C loaded are illustrated below as an example. The subscripts indicate the bearing location and the lane loaded to obtain the respective reaction:

$$R_{5\_c} := \frac{P_{wheel} \cdot (4.25 + 10.25) + W_{lane} \cdot 10 \times 7.25}{9.75} \qquad \boxed{R_{5\_c} = 179.4} \text{ kips}$$

$$R_{4\_c} := P_{wheel} \cdot 2 + W_{lane} \cdot 10 - R_{5\_c} \qquad \boxed{R_{4\_c} = 61.86} \text{ kips}$$

The reactions at bearings 1, 2 and 3 with only Lane C loaded are zero. Calculations similar to those above yield the following live load reactions with the remaining lanes loaded. All reactions shown are in kips.

Lane A Loaded	Lane B Loaded	Lane C Loaded
$R_{5\_a} := 0.0$	$R_{5\_b} := 0.0$	$\boxed{R_{5\_c} = 179.4}$
$R_{4\_a} := 0.0$	$\boxed{R_{4\_b} = 123.66}$	$\boxed{R_{4\_c} = 61.86}$
$\boxed{R_{3\_a} = 72.31}$	$\boxed{R_{3\_b} = 117.56}$	$R_{3\_c} := 0.0$
$\boxed{R_{2\_a} = 164.67}$	$R_{2\_b} := 0.0$	$R_{2\_c} := 0.0$
$\boxed{R_{1\_a} = 4.27}$	$R_{1\_b} := 0.0$	$R_{1\_c} := 0.0$

E13-1.6 Compute Other Load Effects

Other load effects that will be considered for this pier design include braking force, wind loads, and temperature loads.

For simplicity, buoyancy, stream pressure, ice loads and earthquake loads are not included in this design example.

E13-1.6.1 Braking Force

Since expansion bearings exist at the abutments, the entire longitudinal braking force is resisted by the pier.

In accordance with **LRFD [3.6.4]**, the braking force per lane is the greater of:

25 percent of the axle weights of the design truck or tandem

5 percent of the axle weights of the design truck plus lane load

5 percent of the axle weights of the design tandem plus lane load

The total braking force is computed based on the number of design lanes in the same direction. It is assumed in this example that this bridge is likely to become one-directional in the future. Therefore, any and all design lanes may be used to compute the governing braking force. Also, braking forces are not increased for dynamic load allowance in accordance with **LRFD [3.6.2.1]**. The calculation of the braking force for a single traffic lane follows:

25 percent of the design truck:

$BRK_{trk} := 0.25 \cdot (32 + 32 + 8)$ $\boxed{BRK_{trk} = 18}$ kips

25 percent of the design tandem:

$BRK_{tan} := 0.25 \cdot (25 + 25)$ $\boxed{BRK_{tan} = 12.5}$ kips

5 percent of the axle weights of the design truck plus lane load:

$BRK_{trk\_lan} := 0.05 \cdot [(32 + 32 + 8) + (0.64 \times 2 \cdot L)]$ $\boxed{BRK_{trk\_lan} = 11.28}$ kips

5 percent of the axle weights of the design tandem plus lane load:

$BRK_{tan\_lan} := 0.05 \cdot [(25 + 25) + (0.64 \times 2 \cdot L)]$ $\boxed{BRK_{tan\_lan} = 10.18}$ kips

Use:

$BRK := \max(BRK_{trk}, BRK_{tan}, BRK_{trk\_lan}, BRK_{tan\_lan})$ $\boxed{BRK = 18}$ kips per lane

LRFD [3.6.4] states that the braking force is applied along the longitudinal axis of the bridge at a distance of six feet above the roadway surface. However, since the skew angle is zero for thi design example and the bearings are assumed incapable of transmitting longitudinal moment, the braking force will be applied at the top of bearing elevation. For bridges with skews, the component of the braking force in the transverse direction would be applied six feet above the roadway surface.

This force may be applied in either horizontal direction (back or ahead station) to cause the maximum force effects. Additionally, the total braking force is typically assumed equally distributed among the bearings:

$$BRK_{brg} := \frac{BRK}{5} \qquad \boxed{BRK_{brg} = 3.6} \quad \text{kips per bearing per lane}$$

The moment arm about the base of the column is:

$$H_{BRK} := H_{col} + H_{cap} + \frac{H_{brng}}{12} \qquad \boxed{H_{BRK} = 26.53} \quad \text{feet}$$

E13-1.6.2 Wind Load on Superstructure

Prior to calculating the wind load on the superstructure, the structure must be checked for aero elastic instability, **LRFD [3.8.3]**. If the span length to width or depth ratio is greater than 30, the structure is considered wind-sensitive and design wind loads should be based on wind tunnel studies.

$$\boxed{L = 120} \quad \text{feet}$$

$$Width := w_{deck} \qquad \boxed{Width = 46.5} \quad \text{feet}$$

$$Depth := H_{super} - H_{par} \qquad \boxed{Depth = 5.79} \quad \text{feet}$$

$$\boxed{\frac{L}{Width} = 2.58} \quad \text{OK} \qquad \boxed{\frac{L}{Depth} = 20.72} \quad \text{OK}$$

Since the span length to width and depth ratios are both less than 30, the structure does not need to be investigated for aero elastic instability.

To compute the wind load on the superstructure, the area of the superstructure exposed to the wind must be defined. For this example, the exposed area is the total superstructure depth multiplied by length tributary to the pier. Due to expansion bearings at the abutment, the transverse length tributary to the pier is not the same as the longitudinal length.

The superstructure depth includes the total depth from the top of the barrier to the bottom of the girder. Included in this depth is any haunch and/or depth due to the deck cross-slope. Once the total depth is known, the wind area can be calculated and the wind pressure applied.

The total depth was previously computed in Section E13-1.1 and is as follows:
$\boxed{H_{super} = 8.46}$ feet

For this two-span bridge example, the tributary length for wind load on the pier in the transverse direction is one-half of each span:

$$L_{windT} := \frac{L + L}{2} \qquad \boxed{L_{windT} = 120} \quad \text{feet}$$

In the longitudinal direction, the tributary length is the entire bridge length due to the expansion bearings at the abutments:

$$L_{windL} := L \cdot 2 \qquad \boxed{L_{windL} = 240} \quad \text{feet}$$

The transverse wind area is:

$$A_{wsuperT} := H_{super} \cdot L_{windT} \qquad \boxed{A_{wsuperT} = 1015} \quad ft^2$$

The longitudinal wind area is:

$$A_{wsuperL} := H_{super} \cdot L_{windL} \qquad \boxed{A_{wsuperL} = 2031} \quad ft^2$$

In accordance with Section 13.4.4, the design wind velocity, V_B is equal to 100 mph and the design wind pressure does not need to be adjusted. The design wind pressures, per Section 13.4.4.1, are as follows and shall be applied simultaneously:

$$Psup_{trans} := 0.050 \quad \text{ksf}$$

$$Psup_{longit} := 0.019 \quad \text{ksf}$$

Also, the minimum transverse normal wind loading on girders must be greater than or equal to 0.30 klf:

$$Wind_{total} := Psup_{trans} H_{super} \qquad \boxed{Wind_{total} = 0.42} \quad \text{klf}$$

which is greater than 0.30 klf

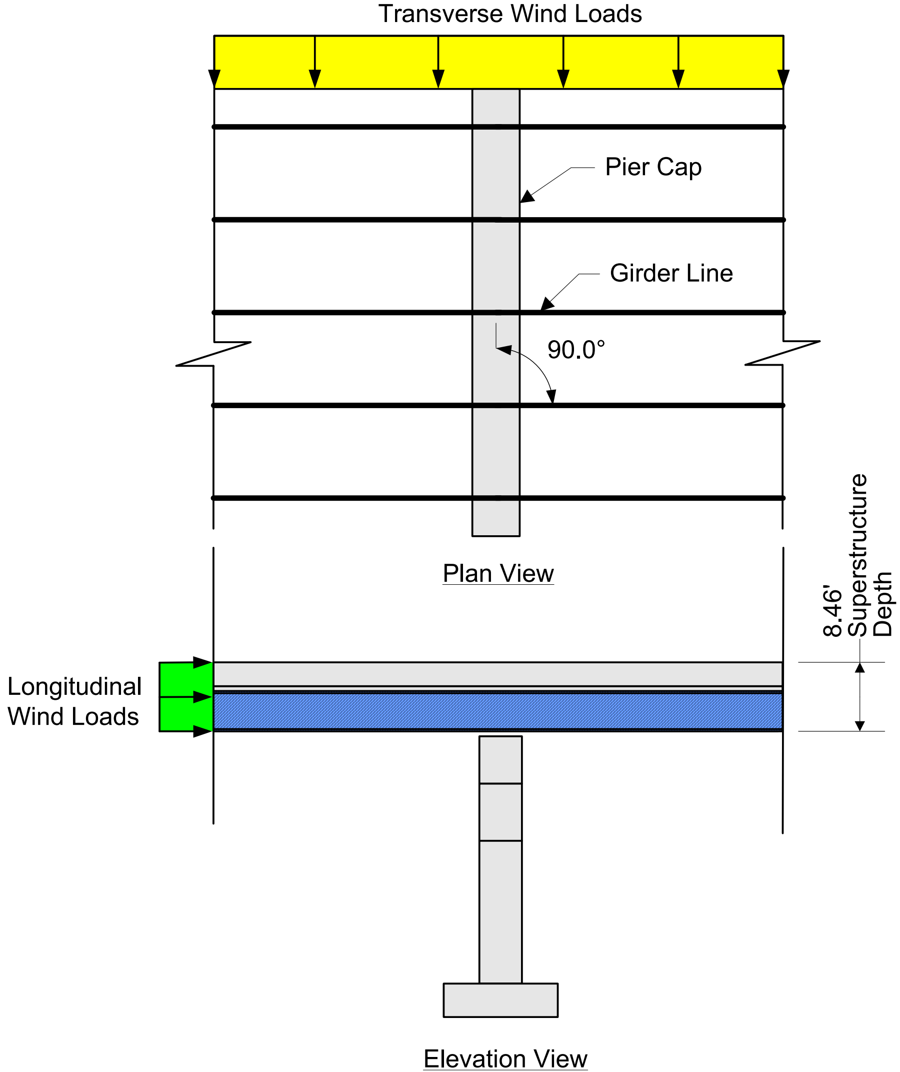

Figure E13-1.6-1
Application of Wind Load

The superstructure wind loads acting on the pier (girders) are:

$WS_{suptrns} := A_{wsuperT} \cdot Psup_{trans}$ $\boxed{WS_{suptrns} = 50.77}$ kips

$WS_{suplng} := A_{wsuperL} \cdot Psup_{longit}$ $\boxed{WS_{suplng} = 38.59}$ kips

The total longitudinal wind load shown above is assumed to be divided equally among the bearings. In addition, the load at each bearing is assumed to be applied at the top of the bearing. These assumptions are consistent with those used in determining the bearing forces due to the longitudinal braking force.

The transverse wind loads shown above are also assumed to be equally divided among the bearings but are applied at the mid-height of the superstructure.

For calculating the resulting moment effect on the column, the moment arm about the base of the column is:

$$H_{WSlong} := H_{col} + H_{cap} + \frac{H_{brng}}{12} \qquad \boxed{H_{WSlong} = 26.53} \quad \text{feet}$$

$$H_{WStrns} := H_{col} + H_{cap} + \frac{H_{brng}}{12} + \frac{H_{super}}{2} \qquad \boxed{H_{WStrns} = 30.76} \quad \text{feet}$$

However, the transverse load also applies a moment to the pier cap. This moment, which acts about the centerline of the pier cap, induces vertical loads at the bearings as illustrated in Figure E13-1.6-2. The computations for these vertical forces are presented below.

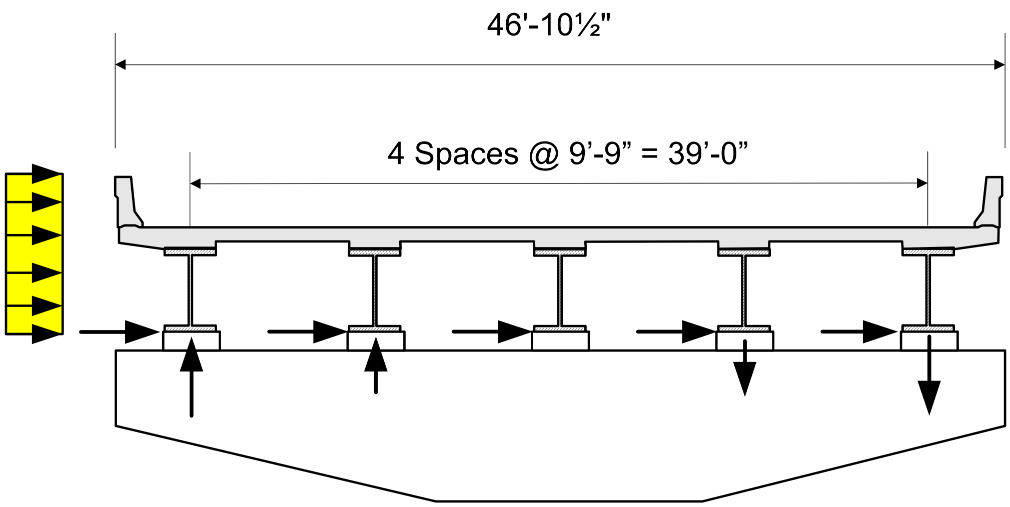

Figure E13-1.6-2
Transverse Wind Loads at Pier Bearings from Wind on Superstructure

$$M_{trns} := WS_{suptrns} \cdot \left(\frac{H_{super}}{2}\right) \qquad \boxed{M_{trns} = 214.8} \quad \text{kip-ft}$$

Moment of Inertia for the Girder Group:

$$I = \Sigma A \cdot y^2$$

$A = 1 \quad I_1 = I_5 \quad I_2 = I_4 \quad I_3 = 0$

$$I_{girders} := 2 \cdot (S + S)^2 + 2 \cdot S^2$$

$$= 2 \cdot (9.75 + 9.75)^2 + 2 \cdot 9.75^2 \qquad \boxed{I_{girders} = 950.63} \quad \text{ft}^2$$

$$\text{Reaction} = \frac{\text{Moment} \cdot y}{I}$$

$$RWS1\_5_{trns} := \frac{M_{trns} \cdot (S + S)}{I_{girders}}$$ $\boxed{RWS1\_5_{trns} = 4.41}$ kips

The loads at bearings 1 and 5 are equal but opposite in direction. Similarly for bearings 2 and 4:

$$RWS2\_4_{trns} := \frac{M_{trns} \cdot S}{I_{girders}}$$ $\boxed{RWS2\_4_{trns} = 2.2}$ kips

Finally, by inspection: $\boxed{RWS3_{trns} = 0}$ kips

E13-1.6.2.1 Vertical Wind Load

The vertical (upward) wind load is calculated by multiplying a 0.020 ksf vertical wind pressure by the out-to-out bridge deck width. It is applied at the windward quarter-point of the deck only for limit states that do not include wind on live load.

From previous definitions:

$\boxed{w_{deck} = 46.5}$ ft $\boxed{L_{windT} = 120}$ ft

The total vertical wind load is then:

$$WS_{vert} := 0.02(w_{deck}) \cdot (L_{windT})$$ $\boxed{WS_{vert} = 111.6}$ kips

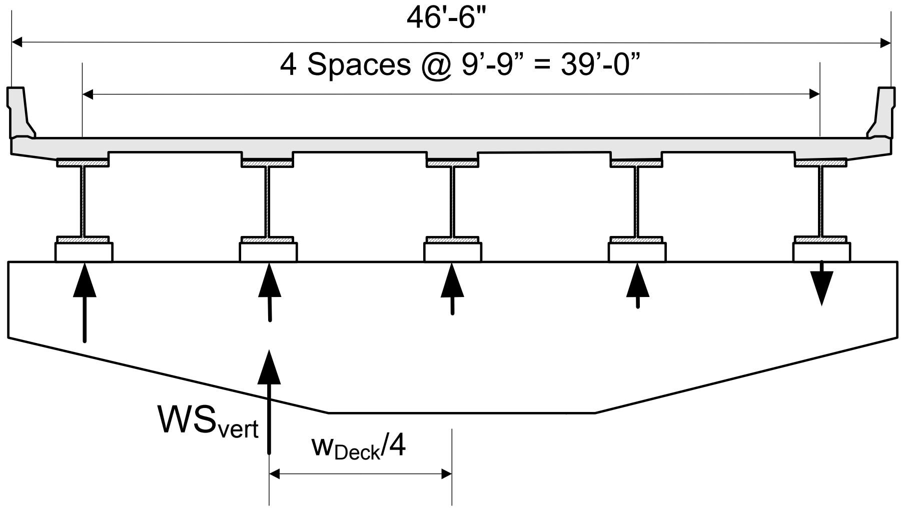

Figure E13-1.6-3
Vertical Wind Loads at Pier Bearings from Wind on Superstructure

This load causes a moment about the pier centerline. The value of this moment is:

$$M_{WS\_vert} := WS_{vert} \cdot \frac{Width}{4}$$ $\boxed{M_{WS\_vert} = 1297.35}$ kip-ft

The loads at the bearings are computed as follows:

$$RWS_{vert1} := \frac{-WS_{vert}}{5} + \frac{M_{WS\_vert} \cdot (2 \cdot S)}{I_{girders}}$$ $\boxed{RWS_{vert1} = 4.29}$ kips

$$RWS_{vert2} := \frac{-WS_{vert}}{5} + \frac{M_{WS\_vert} \cdot S}{I_{girders}}$$ $\boxed{RWS_{vert2} = -9.01}$ kips

$$RWS_{vert3} := \frac{-WS_{vert}}{5}$$ $\boxed{RWS_{vert3} = -22.32}$ kips

$$RWS_{vert4} := \frac{-WS_{vert}}{5} - \frac{M_{WS\_vert} \cdot S}{I_{girders}}$$ $\boxed{RWS_{vert4} = -35.63}$ kips

$$RWS_{vert5} := \frac{-WS_{vert}}{5} - \frac{M_{WS\_vert} \cdot 2 \cdot S}{I_{girders}}$$ $\boxed{RWS_{vert5} = -48.93}$ kips

Where a negative value indicates a vertical upward load.

E13-1.6.2.2 Wind Load on Vehicles

The representation of wind pressure acting on vehicular traffic is given by the Specifications as a uniformly distributed load. This load is applied both transversely and longitudinally. For the transverse and longitudinal loadings, the total force in each respective direction is calculated by multiplying the appropriate component by the length of structure tributary to the pier. Similar to the superstructure wind loading, the longitudinal length tributary to the pier differs from the transverse length. In accordance with Section 13.4.4.3 the transverse and longitudinal loads shown below shall be applied simultaneously. Also see 13.5.

$\boxed{L_{windT} = 120}$ feet $\boxed{L_{windL} = 240}$ feet

$P_{LLtrans} := 0.100$ klf

$P_{LLlongit} := 0.040$ klf

$$WL_{trans} := L_{windT} \cdot P_{LLtrans}$$ $\boxed{WL_{trans} = 12}$ kips

$$WL_{long} := L_{windL} \cdot P_{LLlongit}$$ $\boxed{WL_{long} = 9.6}$ kips

The wind on vehicular live loads shown above are applied to the bearings in the same manner as the wind load from the superstructure. That is, the total transverse and longitudinal load is equally distributed to each bearing and applied at the the top of the bearing. In addition, the transverse load acting six feet above the roadway applies a moment to the pier cap. This moment induces vertical reactions at the bearings. The values of these vertical reactions are given below. The computations for these reactions are not shown but are carried out as shown in the subsection "Wind Load from Superstructure." The only difference is that the moment arm used for calculating the moment is equal to (H_{super} - H_{par} + 6.0 feet).

$\boxed{RWL1\_5_{trns} = 2.9}$ kips

$\boxed{RWL2\_4_{trns} = 1.45}$ kips

$\boxed{RWL3_{trns} = 0}$ kips

For calculating the resulting moment effect on the column, the moment arm about the base of the column is:

$$H_{WLlong} := H_{col} + H_{cap} + \frac{H_{brng}}{12} \qquad \boxed{H_{WLlong} = 26.53} \text{ feet}$$

$$H_{WLtrns} := H_{col} + H_{cap} + \frac{H_{brng}}{12} + \left(H_{super} - H_{par} + 6\right) \qquad \boxed{H_{WLtrns} = 38.32} \text{ feet}$$

E13-1.6.3 Wind Load on Substructure

The Specifications state that the wind loads acting directly on substructure units shall be calculated from a base wind pressure of 0.040 ksf. In accordance with Section 13.4.4.2, these loads are applied simultaneously in the transverse and longitudinal directions of the pier. These loads act simultaneously with the superstructure wind loads.

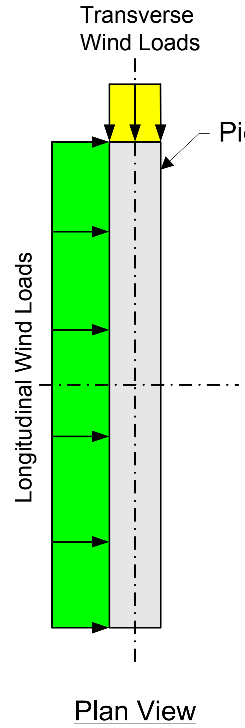

Figure E13-1.6-4
Wind Pressure on Pier

What follows is an example of the calculation of the wind loads acting directly on the pier. For simplicity, the tapers of the pier cap overhangs will be considered solid. The column height exposed to wind is the distance from the ground line (which is two feet above the footing) to the bottom of the pier cap.

Component areas of the pier cap:

$$A_{capLong} := (L_{cap}) \cdot (H_{cap}) \qquad \boxed{A_{capLong} = 511.5} \quad ft^2$$

$$A_{capTrans} := (W_{cap}) \cdot (H_{cap}) \qquad \boxed{A_{capTrans} = 44} \quad ft^2$$

Component areas of the pier column:

$$A_{colLong} := (L_{col}) \cdot (H_{col} - D_{soil}) \qquad \boxed{A_{colLong} = 201.5} \quad ft^2$$

$$A_{colTrans} := (W_{col}) \cdot (H_{col} - D_{soil}) \qquad \boxed{A_{colTrans} = 52} \quad ft^2$$

The transverse and longitudinal force components are:

$P_{sub} := 0.040$ ksf

$WS_{subL} := P_{sub} \cdot (A_{capLong} + A_{colLong})$ $\boxed{WS_{subL} = 28.52}$ kips

$WS_{subT} := P_{sub} \cdot (A_{capTrans} + A_{colTrans})$ $\boxed{WS_{subT} = 3.84}$ kips

The point of application of these loads will be the centroid of the loaded area of each face, respectively.

$$H_{WSsubL} := \frac{A_{capLong} \cdot \left(H_{col} + \frac{H_{cap}}{2}\right) + A_{colLong} \cdot \left(\frac{H_{col} - 2}{2} + 2\right)}{A_{capLong} + A_{colLong}}$$

$\boxed{H_{WSsubL} = 17.11}$ feet

$$H_{WSsubT} := \frac{A_{capTrans} \cdot \left(H_{col} + \frac{H_{cap}}{2}\right) + A_{colTrans} \cdot \left(\frac{H_{col} - 2}{2} + 2\right)}{A_{capTrans} + A_{colTrans}}$$

$\boxed{H_{WSsubT} = 14}$ feet

E13-1.6.4 Temperature Loading (Superimposed Deformations)

In this particular structure, with a single pier centered between two abutments that have identical bearing types, the temperature force is based on assuming a minimum coefficient of expansion at one abutment and the maximum at the other using only dead load reactions. This force acts in the longitudinal direction of the bridge (either back or ahead station) and is equally divided among the bearings. Also, the forces at each bearing from this load will be applied at the top of the bearing.

The abutment girder Dead Load reactions from E13-1.1.3.1 are as follows:

$\boxed{AbutRint_{DC} = 61.69}$ $\boxed{AbutRext_{DC} = 62.97}$

$\boxed{AbutRint_{DW} = 7.46}$ $\boxed{AbutRext_{DW} = 7.46}$

$\boxed{\mu_{min} = 0.06}$

$\boxed{\mu_{max} = 0.1}$

$\Delta\mu := \mu_{max} - \mu_{min}$ $\boxed{\Delta\mu = 0.04}$

$$F_{TU} := \Delta\mu \cdot \left[3 \cdot (AbutRint_{DC} + AbutRint_{DW}) + 2 \cdot (AbutRext_{DC} + AbutRext_{DW})\right]$$

$\boxed{F_{TU} = 13.93}$ kips

The resulting temperature force acting on each bearing is:

$$TU_{BRG} := \frac{F_{TU}}{5} \qquad \boxed{TU_{BRG} = 2.79} \quad \text{kips}$$

The moment arm about the base of the column is:

$$H_{TU} := H_{col} + H_{cap} + \frac{H_{brng}}{12} \qquad \boxed{H_{TU} = 26.53} \quad \text{feet}$$

E13-1.7 Analyze and Combine Force Effects

The first step within this design step will be to summarize the loads acting on the pier at the bearing locations. This is done in Tables E13-1.7-1 through E13-1.7-8 shown below. Tables E13-1.7-1 through E13-1.7-5 summarize the vertical loads, Tables E13-1.7-6 through E13-1.7-7 summarize the horizontal longitudinal loads, and Table E13-1.7-8 summarizes the horizontal transverse loads. These loads along with the pier self-weight loads, which are shown after the tables, need to be factored and combined to obtain total design forces to be resisted in the pier cap, column and footing.

It will be noted here that loads applied due to braking and temperature can act either ahead or back station. Also, wind loads can act on either side of the structure and with positive or negative skew angles. This must be kept in mind when considering the signs of the forces in the tables below. The tables assume a particular direction for illustration only.

	Superstructure Dead Load		Wearing Surface Dead Load	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	$Rext_{DC}$	246.65	$Rext_{DW}$	27.32
2	$Rint_{DC}$	241.72	$Rext_{DW}$	27.32
3	$Rint_{DC}$	241.72	$Rext_{DW}$	27.32
4	$Rint_{DC}$	241.72	$Rext_{DW}$	27.32
5	RextDC	246.65	$Rext_{DW}$	27.32

Table E13-1.7-1
Unfactored Vertical Bearing Reactions from Superstructure Dead Load

	Vehicular Live Load **					
	Lane A		Lane B		Lane C	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	R_1_a	4.27	R_1_b	0.00	R_1_c	0.00
2	R_2_a	164.67	R_2_b	0.00	R_2_c	0.00
3	R_3_a	72.31	R_3_b	117.56	R_3_c	0.00
4	R_4_a	0.00	R_4_b	123.66	R_4_c	61.86
5	R_5_a	0.00	R_5_b	0.00	R_5_c	179.40

**Note: Live load reactions include impact on truck loading.

Table E13-1.7-2
Unfactored Vertical Bearing Reactions from Live Load

Bearing No.	Reactions from Transverse Wind Load on Superstructure (kips)
1	4.41
2	2.20
3	0.00
4	-2.20
5	-4.41

Table E13-1.7-3
Unfactored Vertical Bearing Reactions from Wind on Superstructure

Bearing No.	Reactions from Transverse Wind Load on Vehicular Live Load (kips)
1	2.90
2	1.45
3	0.00
4	-1.45
5	-2.90

Table E13-1.7-4
Unfactored Vertical Bearing Reactions from Wind on Live Load

	Vertical Wind Load on Superstructure	
Bearing No.	Variable Name	Reaction (Kips)
1	RWS_{vert1}	4.29
2	RWS_{vert2}	-9.01
3	RWS_{vert3}	-22.32
4	RWS_{vert4}	-35.63
5	RWS_{vert5}	-48.93

Table E13-1.7-5
Unfactored Vertical Bearing Reactions from Vertical Wind on Superstructure

	Braking Load **		Temperature Loading	
	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
Each Bearing	BRK_{brg}	3.60	TU_1	2.79

**Note: Values shown are for a single lane loaded

Table E13-1.7-6
Unfactored Horizontal Longitudinal Bearing Reactions from Braking and Temperature

Load Type	Unfactored Horizontal Longitudinal Forces (kips)
Wind Loads from Superstructure	38.59
Wind on Live Load	9.60
Wind on Pier	28.52

Table E13-1.7-7
Unfactored Horizontal Longitudinal Forces

Load Type	Unfactored Horizontal Transverse Forces (kips)
Wind Loads from Superstructure	50.77
Wind on Live Load	12.00
Wind on Pier	3.84

Table E13-1.7-8
Unfactored Horizontal Transverse Forces

In addition to all the loads tabulated above, the pier self-weight must be considered when determining the final design forces. Additionally for the footing and pile designs, the weight of the earth on top of the footing must be considered. These loads were previously calculated and are shown below:

$\boxed{DL_{Cap} = 251.1}$ kips $\boxed{DL_{ftg} = 144.9}$ kips

$\boxed{DL_{col} = 139.5}$ kips $\boxed{EV_{ftg} = 51.36}$ kips

In the AASHTO LRFD design philosophy, the applied loads are factored by statistically calibrated load factors. In addition to these factors, one must be aware of two additional sets of factors which may further modify the applied loads.

The first set of additional factors applies to all force effects and are represented by the Greek letter h (eta) in the Specifications, **LRFD [1.3.2.1]**. These factors are related to the ductility, redundancy, and operational importance of the structure. A single, combined eta is required for every structure. In accordance with WisDOT policy, all eta factors are taken equal to one.

The other set of factors mentioned in the first paragraph above applies only to the live load force effects and are dependent upon the number of loaded lanes. These factors are termed multiple presence factors by the Specifications, **LRFD [T3.6.1.1.2-1]**. These factors for this bridge are shown as follows:

Multiple presence factor, m (1 lane) $m_1 := 1.20$

Multiple presence factor, m (2 lanes) $m_2 := 1.00$

Multiple presence factor, m (3 lanes) $m_3 := 0.85$

Table E13-1.7-9 contains the applicable limit states and corresponding load factors that will be used for this pier design. Limit states not shown either do not control the design or are not applicable. The load factors shown in Table E13-1.7-9 are the standard load factors assigned by the Specifications and are exclusive of multiple presence and eta factors.

It is important to note here that the maximum load factors shown in Table E13-1.7-9 for uniform temperature loading (TU) apply only for deformations, and the minimum load factors apply for all other effects. Since the force effects from the uniform temperature loading are considered in this pier design, the minimum load factors will be used.

	Load Factors							
	Strength I		Strength III		Strength V		Service I	
Load	γ max	γ min	γ max	γ min	γ max	γ min	γ max	γ min
DC	1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.00
DW	1.50	0.65	1.50	0.65	1.50	0.65	1.00	1.00
LL	1.75	1.75	---	---	1.35	1.35	1.00	1.00
BR	1.75	1.75	---	---	1.35	1.35	1.00	1.00
TU	1.20	0.50	1.20	0.50	1.20	0.50	1.20	1.00
WS	---	---	1.40	1.40	0.40	0.40	0.30	0.30
WL	---	---	---	---	1.00	1.00	1.00	1.00
EV	1.35	1.00	1.35	1.00	1.35	1.00	1.00	1.00

Table E13-1.7-9
Load Factors and Applicable Pier Limit States

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states in the pier cap, column, footing and piles. Design calculations will be carried out for the governing limit states only.

E13-1.7.1 Pier Cap Force Effects

The pier cap will be designed using a strut and tie model. See E13-1.8 for additional information. For this type of model, the member's self weight is included in the bearing reactions. The calculation of the Strength 1 Factored girder reactions follows.

For the dead load of the cap, the tributary weight of the cap will be added to each girder reaction.

$$Cap_{DC\_1} := 8.625 \cdot \frac{5 + 8.34}{2} \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_1} = 34.52} \text{ kips}$$

$$Cap_{DC\_2} := \left(6.875 \cdot \frac{8.34 + 11}{2} + 2.875 \cdot 11\right) \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_2} = 58.86} \text{ kips}$$

$$Cap_{DC\_3} := 9.75 \cdot 11 \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_3} = 64.35} \text{ kips}$$

$$Cap_{DC\_4} := Cap_{DC\_2} \qquad \boxed{Cap_{DC\_4} = 58.86} \text{ kips}$$

$$Cap_{DC\_5} := Cap_{DC\_1} \qquad \boxed{Cap_{DC\_5} = 34.52} \text{ kips}$$

Look at the combined live load girder reactions with 1 (Lane C), 2 (Lanes C and B) and 3 lanes (Lanes C, B and A) loaded. The multiple presence factor from E13-1.7 shall be applied. The design lane locations were located to maximize the forces over the right side of the cap.

	Unfactored Vehicular Live Load					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	R_1_1	0.00	R_1_2	0.00	R_1_3	3.63
2	R_2_1	0.00	R_2_2	0.00	R_2_3	139.97
3	R_3_1	0.00	R_3_b	117.56	R_3_3	161.40
4	R_4_1	74.23	R_4_2	185.52	R_4_3	157.70
5	R_5_1	215.27	R_5_b	179.40	R_5_3	152.49

Table E13-1.7-10

Unfactored Vehicular Live Load Reactions

Calculate the Strength 1 Combined Girder Reactions for 1, 2 and 3 lanes loaded. An example calculation is shown for the girder 5 reaction with one lane loaded. Similar calculations are performed for the remaining girders and number of lanes loaded.

$$Ru_{5\_1} := \gamma_{DCmax} \cdot \left(Rext_{DC} + Cap_{DC\_5}\right) + \gamma_{DWmax} \cdot Rext_{DW} + \gamma_{LL} \cdot R_{5\_1}$$

$$\boxed{Ru_{5\_1} = 769.17} \quad \text{kips}$$

	Total Factored Girder Reactions**					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	Ru_1_1	392.44	Ru_1_2	392.44	Ru_1_3	398.79
2	Ru_2_1	416.71	Ru_2_2	416.71	Ru_2_3	661.66
3	Ru_3_1	423.57	Ru_3_b	629.30	Ru_3_3	706.01
4	Ru_4_1	546.62	Ru_4_2	741.38	Ru_4_3	692.68
5	Ru_5_1	769.17	Ru_5_b	706.38	Ru_5_3	659.29

** Includes dead load of pier cap

Table E13-1.7-11

Factored Girder Reactions for STM Cap Design

E13-1.7.2 Pier Column Force Effects

The controlling limit states for the design of the pier column are Strength V (for biaxial bending with axial load). The critical design location is where the column meets the footing, or at the column base. The governing force effects for Strength V are achieved by minimizing the axial effects while maximizing the transverse and longitudinal moments. This is accomplished by excluding the future wearing surface, applying minimum load factors on the structure dead load, and loading only Lane B and Lane C with live load.

For Strength V, the factored vertical forces and corresponding moments at the critical section are shown below.

Strength V Axial Force:

$\boxed{Rext_{DC} = 246.65}$ kips

$\boxed{Rint_{DC} = 241.72}$ kips

$\boxed{DL_{Cap} = 251.1}$ kips

$\boxed{DL_{col} = 139.5}$ kips

$\boxed{WS_{vert} = 111.6}$ kips (uplift)

w_{cap}

$\boxed{R_{3\_2} = 117.56}$ kips

$\boxed{R_{4\_2} = 185.52}$ kips

$\boxed{R_{5\_2} = 179.4}$ kips

$$Ax_{colStrV} := \gamma_{DCminStrV} \cdot (2 \cdot Rext_{DC} + 3 \cdot Rint_{DC} + DL_{Cap} + DL_{col}) \ldots \\ + \gamma_{LLStrV}(R_{3\_2} + R_{4\_2} + R_{5\_2}) \ldots \\ + -\gamma_{WSStrV} \cdot WS_{vert}$$

$\boxed{Ax_{colStrV} = 2054.87}$ kips

Strength V Transverse moment:

$ArmV3_{col} := 0$ $\boxed{ArmV3_{col} = 0}$ feet

$ArmV4_{col} := S$ $\boxed{ArmV4_{col} = 9.75}$ feet

$ArmV5_{col} := 2 \cdot S$ $\boxed{ArmV5_{col} = 19.5}$ feet

$\boxed{WS_{suptrns} = 50.77}$ kips $\boxed{H_{WStrns} = 30.76}$ feet

$\boxed{WL_{trans} = 12}$ kips $\boxed{H_{WLtrns} = 38.32}$ feet

$\boxed{WS_{subT} = 3.84}$ kips $\boxed{H_{WSsubT} = 14}$ feet

$\boxed{M_{WS\_vert} = 1297.35}$ kip-ft

$$MuT_{colStrV} := \gamma_{LLStrV}\left(R_{3\_2}\cdot ArmV3_{col} + R_{4\_2}\cdot ArmV4_{col} + R_{5\_2}\cdot ArmV5_{col}\right) \ldots$$
$$+ \gamma_{WLStrV}\cdot\left(WL_{trans}\cdot H_{WLtrns}\right) \ldots$$
$$+ \gamma_{WSStrV}\cdot\left(M_{WS\_vert} + WS_{suptrns}\cdot H_{WStrns} + WS_{subT}\cdot H_{WSsubT}\right)$$

$\boxed{MuT_{colStrV} = 8789.59}$ kip-ft

Strength V Longitudinal moment:

$BRK_{brg} = 3.6$	kips/bearing per lane	$H_{BRK} = 26.53$	feet
$TU_{BRG} = 2.79$	kips/ bearing	$H_{TU} = 26.53$	feet
$WS_{suplng} = 38.59$	kips	$H_{WSlong} = 26.53$	feet
$WL_{long} = 9.6$	kips	$H_{WLlong} = 26.53$	feet
$WS_{subL} = 28.52$	kips	$H_{WSsubL} = 17.11$	feet
$m_2 = 1.00$	multi presence factor for two lanes loaded		

$$MuL_{colStrV} := \gamma_{BRStrV}\cdot\left(5\cdot BRK_{brg}\cdot H_{BRK}\cdot 2\cdot m_2\right) \ldots$$
$$+ \gamma_{TUminStrV}\left(5TU_{BRG}\cdot H_{TU}\right) \ldots$$
$$+ \gamma_{WLStrV}\cdot\left(WL_{long}\cdot H_{WLlong}\right) \ldots$$
$$+ \gamma_{WSStrV}\cdot\left(WS_{suplng}\cdot H_{WSlong} + WS_{subL}\cdot H_{WSsubL}\right)$$

$\boxed{MuL_{colStrV} = 2333.6}$ kip-ft

For Strength III, the factored transverse shear in the column is:

$\boxed{WS_{subT} = 3.84}$ kips $\boxed{WS_{suptrns} = 50.77}$ kips

$VuT_{col} := \gamma_{WSStrIII}\left(WS_{suptrns} + WS_{subT}\right)$ $\boxed{VuT_{col} = 76.45}$ kips

For Strength V, the factored longitudinal shear in the column is (reference Table E13-1.7-7):

$\boxed{WL_{long} = 9.6}$ $\boxed{WS_{subL} = 28.52}$ $\boxed{WS_{suplng} = 38.59}$ kips

$$VuL_{col} := \gamma_{WSStrV}\left(WS_{suplng} + WS_{subL}\right) + \gamma_{WLStrV}\cdot WL_{long} \ldots$$
$$+ \gamma_{TUmin}\cdot\left(TU_{BRG}\cdot 5\right) + \gamma_{BRStrV}\cdot\left(5\cdot BRK_{brg}\right)\cdot 3\cdot m_3$$

$\boxed{VuL_{col} = 105.37}$ kips

E13-1.7.3 Pier Pile Force Effects

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The force effects in the piles cannot be determined without a pile layout. The pile layout depends upon the pile capacity and affects the footing design. The pile layout used for this pier foundation is shown in Figure E13-1.10-1.

Based on the pile layout shown in Figure E13-1.10-1, the controlling limit states for the pile design are Strength I (for maximum pile load), Strength III (for minimum pile load), and Strength V (for maximum horizontal loading of the pile group).

Structure Dead Load Effects

Girder DC Reactions:

$\boxed{Rext_{DC} = 246.65}$ kips

$\boxed{Rint_{DC} = 241.72}$ kips

Girder DW Reactions:

$\boxed{Rext_{DW} = 27.32}$ kips

$\boxed{Rint_{DW} = 27.32}$ kips

$DC_{Super} := 2 \cdot Rext_{DC} + 3 \cdot Rint_{DC}$ $\boxed{DC_{Super} = 1218.46}$ kips

$\boxed{DL_{Cap} = 251.1}$ kips

$\boxed{DL_{col} = 139.5}$ kips

$\boxed{DL_{ftg} = 144.9}$ kips

$DC_{pile} := DC_{Super} + DL_{Cap} + DL_{col} + DL_{ftg}$ $\boxed{DC_{pile} = 1753.96}$ kips

$DW_{pile} := 2 \cdot Rext_{DW} + 3 \cdot Rint_{DW}$ $\boxed{DW_{pile} = 136.6}$ kips

Vertical Earth Load Effects

$EV_{pile} := EV_{ftg}$ $\boxed{EV_{pile} = 51.36}$ kips

Live Load Effects (without Dynamic Load Allowance)

<u>Live Load Girder Reactions for 2 lanes, Lanes B and C, loaded:</u>

$\boxed{R_{1\_2p} = 0}$ kips

$\boxed{R_{2\_2p} = 0}$ kips

$\boxed{R_{3\_2p} = 99.21}$ kips

$\boxed{R_{4\_2p} = 156.54}$ kips

$\boxed{R_{5\_2p} = 151.38}$ kips $\boxed{R_{T\_2p} = 407.13}$ kips

From Section E13-1.7, the Transverse moment arm for girders 3, 4 and 5 are:

$\boxed{ArmV3_{col} = 0}$ feet

$\boxed{ArmV4_{col} = 9.75}$ feet

$\boxed{ArmV5_{col} = 19.5}$ feet

The resulting Transverse moment applied to the piles is:

$$M_{LL2T\_p} := R_{3\_2p} \cdot ArmV3_{col} + R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}$$

$\boxed{M_{LL2T\_p} = 4478.2}$ kip-ft

The Longitudinal Strength 1 Moment includes the breaking and temperature forces.

$$MuL2_{colStr1} := \gamma_{BR} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) + \gamma_{TUmin}\left(5TU_{BRG} \cdot H_{TU}\right)$$

$\boxed{MuL2_{colStr1} = 1856.29}$ kip-ft

Strength 1 Load for Maximum Pile Reaction

The maximum pile load results from the Strength I load combination with two lanes loaded.

$$Pu2_{pile\_Str1} := \gamma_{DCmax} \cdot DC_{pile} + \gamma_{DWmax} \cdot DW_{pile} + \gamma_{EVmax} \cdot EV_{pile} + \gamma_{LL} \cdot R_{T\_2p}$$

$\boxed{Pu2_{pile\_Str1} = 3179.17}$ kips

$MuT2_{pile\_Str1} := \gamma_{LL} \cdot M_{LL2T\_p}$ $\boxed{MuT2_{pile\_Str1} = 7836.85}$ kip-ft

$MuL2_{pile\_Str1} := MuL2_{colStr1}$ $\boxed{MuL2_{pile\_Str1} = 1856.29}$ kip-ft

Minimum Load on Piles Strength V

The calculation for the minimum axial load on piles is similar to the Strength V axial column loac calculated previously. The weight of the footing and soil surcharge are included. The girder reactions used for pile design do not include impact. The DW loads are not included.

$$\begin{aligned} Pu_{pile\_StrV} := {} & \gamma_{DCminStrV} \cdot \left(2 \cdot Rext_{DC} + 3 \cdot Rint_{DC} + DL_{Cap} + DL_{col} + DL_{ftg}\right) \ldots \\ & + \gamma_{EVminStrV} \cdot EV_{pile} \ldots \\ & + \gamma_{LLStrV}\left(R_{3\_2p} + R_{4\_2p} + R_{5\_2p}\right) \ldots \\ & + -\gamma_{WSStrV} \cdot WS_{vert} \end{aligned}$$

$\boxed{Pu_{pile\_StrV} = 2134.91}$ kips

The calculation for the Strength V longitudinal moment is the same as the longitudinal moment on the column calculated previously. These loads include the breaking force, temperature, wind on live load and wind on the structure.

$$MuL_{pile\_StrV} := \gamma_{BRStrV} \cdot (5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2) \ldots$$
$$+ \gamma_{TUminStrV}(5TU_{BRG} \cdot H_{TU}) \ldots$$
$$+ \gamma_{WLStrV} \cdot (WL_{long} \cdot H_{WLlong}) \ldots$$
$$+ \gamma_{WSStrV} \cdot (WS_{suplng} \cdot H_{WSlong} + WS_{subL} \cdot H_{WSsubL})$$

$$\boxed{MuL_{pile\_StrV} = 2333.6} \text{ kip-ft}$$

The calculation for the Strength V transverse moment is the similar as the transverse moment on the column calculated previously. These loads include the live load, wind on live load and wind on the structure. Impact is not included in these live load reactions.

$$MuT_{pile\_StrV} := \gamma_{LLStrV}(R_{3\_2p} \cdot ArmV3_{col} + R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}) \ldots$$
$$+ \gamma_{WLStrV} \cdot (WL_{trans} \cdot H_{WLtrns}) \ldots$$
$$+ \gamma_{WSStrV} \cdot (M_{WS\_vert} + WS_{suptrns} \cdot H_{WStrns} + WS_{subT} \cdot H_{WSsubT})$$

$$\boxed{MuT_{pile\_StrV} = 7670.61} \text{ kip-ft}$$

For Strength III, the factored transverse shear in the footing is equal to the transverse force at the base of the column.

$$HuT_{pileStrIII} := VuT_{col}$$

$$= \gamma_{WSStrIII} \cdot (WS_{suptrns} + WS_{subT}) \qquad \boxed{HuT_{pileStrIII} = 76.45} \text{ kips}$$

For Strength V, the factored longitudinal shear in the column is equal to the longitudinal force at the base of the column.

$$HuL_{pileStrV} := VuL_{col} \qquad \boxed{HuL_{pileStrV} = 105.37} \text{ kips}$$

The following is a summary of the controlling forces on the piles:

Strength I

$$\boxed{Pu2_{pile\_Str1} = 3179.17} \text{ kips}$$

$$\boxed{MuT2_{pile\_Str1} = 7836.85} \text{ kip-ft}$$

$$\boxed{MuL2_{pile\_Str1} = 1856.29} \text{ kip-ft}$$

Strength III

$\boxed{HuT_{pileStrIII} = 76.45}$ kips

Strength V

$\boxed{Pu_{pile\_StrV} = 2134.91}$ kips

$\boxed{MuT_{pile\_StrV} = 7670.61}$ kip-ft

$\boxed{MuL_{pile\_StrV} = 2333.6}$ kip-ft

$\boxed{HuL_{pileStrV} = 105.37}$ kips

E13-1.7.4 Pier Footing Force Effects

The controlling limit states for the design of the pier footing are **Strength I (for flexure, punching shear at the column, and punching shear at the maximum loaded pile, and for one-way shear).** In accordance with Section 13.11, the footings do not require the crack control by distribution check in **LRFD [5.7.3.4]**. As a result, the Service I Limit State is not required. There is not a single critical design location in the footing where all of the force effects just mentioned are checked. Rather, the force effects act at different locations in the footing and must be checked at their respective locations. For example, the punching shear checks are carried out using critical perimeters around the column and maximum loaded pile, while the flexure and one-way shear checks are carried out on a vertical face of the footing either parallel or perpendicular to the bridge longitudinal axis. Also note that impact is not included for members that are below ground. The weight of the footing concrete and the soil above the footing are not included in these loads as they counteract the pile reactions.

$DC_{ftg} := DC_{Super} + DL_{Cap} + DL_{col}$ $\boxed{DC_{ftg} = 1609.06}$ kips

$DW_{ftg} := 2 \cdot Rext_{DW} + 3 \cdot Rint_{DW}$ $\boxed{DW_{ftg} = 136.6}$ kips

Unfactored Live Load reactions for one, two and three lanes loaded:

$\boxed{R_{T\_1p} = 244.3}$ kips

$\boxed{R_{T\_2p} = 407.13}$ kips

$\boxed{R_{T\_3p} = 519.1}$ kips

The resulting Transverse moment applied to the piles is:

$M_{LL1T} := R_{4\_1p} \cdot ArmV4_{col} + R_{5\_1p} \cdot ArmV5_{col}$ $\boxed{M_{LL1T} = 4153.03}$ kip-ft

$M_{LL2T} := R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}$ $\boxed{M_{LL2T} = 4478.2}$ kip-ft

$$M_{LL3T} := \left(-R_{2\_3p} + R_{4\_3p}\right)\cdot ArmV4_{col} + \left(-R_{1\_3p} + R_{5\_3p}\right)\cdot ArmV5_{col}$$

$$\boxed{M_{LL3T} = 2595.17} \quad \text{kip-ft}$$

The maximum pile load results from the Strength I load combination with two lanes loaded.

$$Pu2_{ftgStr1} := \gamma_{DCmax}\cdot DC_{ftg} + \gamma_{DWmax}\cdot DW_{ftg} + \gamma_{LL}\cdot R_{T\_2p}$$

$$\boxed{Pu2_{ftgStr1} = 2928.7} \quad \text{kips}$$

$$MuT2_{ftgStr1} := \gamma_{LL}\cdot M_{LL2T} \qquad \boxed{MuT2_{ftgStr1} = 7836.85} \quad \text{kip-ft}$$

$$MuL2_{ftgStr1} := \gamma_{BR}\cdot\left(5\cdot BRK_{brg}\cdot H_{BRK}\cdot 2\cdot m_2\right) \ldots + \gamma_{TUmin}\left(5TU_{BRG}\cdot H_{TU}\right) \qquad \boxed{MuL2_{ftgStr1} = 1856.29} \quad \text{kip-ft}$$

The Strength I limit state controls for the punching shear check at the column. In this case the future wearing surface is included, maximum factors are applied to all the dead load components, and all three lanes are loaded with live load. This results in the following bottom of column forces:

$$Pu3_{ftgStr1} := \gamma_{DCmax}\cdot DC_{ftg} + \gamma_{DWmax}\cdot DW_{ftg} + \gamma_{LL}\cdot R_{T\_3p}$$

$$\boxed{Pu3_{ftgStr1} = 3124.66} \quad \text{kips}$$

$$MuT3_{ftgStr1} := \gamma_{LL}\cdot M_{LL3T} \qquad \boxed{MuT3_{ftgStr1} = 4541.55} \quad \text{kip-ft}$$

$$MuL3_{ftgStr1} := \gamma_{BR}\cdot\left(5\cdot BRK_{brg}\cdot H_{BRK}\cdot 3\cdot m_3\right) \ldots + \gamma_{TUmin}\left(5TU_{BRG}\cdot H_{TU}\right) \qquad \boxed{MuL3_{ftgStr1} = 2315.94} \quad \text{kip-ft}$$

E13-1.8 Design Pier Cap - Strut and Tie Model (STM)

Prior to carrying out the actual design of the pier cap, a brief discussion is in order regarding the design philosophy that will be used for the design of the structural components of this pier.

When a structural member meets the definition of a deep component, the Specifications recommends, although does not mandate, that a strut-and-tie model be used to determine force effects and required reinforcing. **LRFD [C5.6.3.1]** indicates that a strut-and-tie model properly accounts for nonlinear strain distribution, nonuniform shear distribution, and the mechanical interaction of V_u, T_u and M_u. Use of strut-and-tie models for the design of reinforced concrete members is new to the LRFD Specification. WisDOT policy is to design hammerhead pier caps using STM.

E13-1.8.1 Determine Geometry and Member Forces

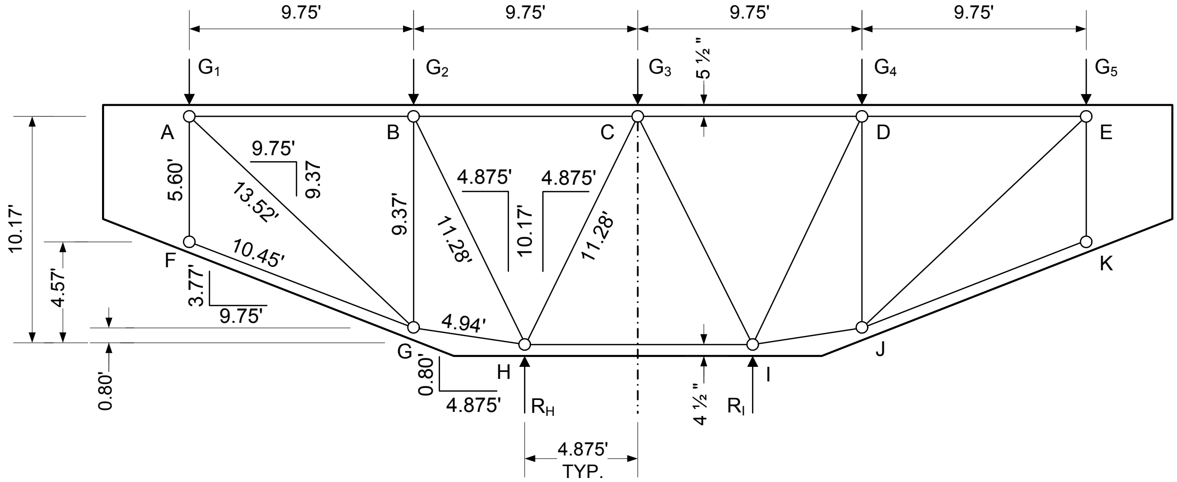

Figure E13-1.8-1
Strut and Tie Model Dimensions

In order to maintain a minimum 25º angle between struts and ties, the support nodes (H and I) are located midway between the girder reactions. For this example a compressive strut depth of 8 inches will be used, making the centroids of the bottom truss chords 4.5 inches from the concrete surface. It is also assumed that two layers of rebar will be required along the top tension ties, and the centroid is located 5.5 inches below the top of the cap.

$centroid_{bot} := 4.5$ inches

$centroid_{top} := 5.5$ inches

Strength I Loads:

	Total Factored Girder Reactions**					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	Ru_1_1	392.44	Ru_1_2	392.44	Ru_1_3	398.79
2	Ru_2_1	416.71	Ru_2_2	416.71	Ru_2_3	661.66
3	Ru_3_1	423.57	Ru_3_2	629.30	Ru_3_3	706.01
4	Ru_4_1	546.62	Ru_4_2	741.38	Ru_4_3	692.68
5	Ru_5_1	769.17	Ru_5_2	706.38	Ru_5_3	659.29

** Includes dead load of pier cap

Table E13-1.8-1
Total Factored Girder Reactions

Calculate the forces in the members for the Strength I Load Case with 2 lanes loaded.

To find the column reaction at node I, sum moments about node H:

$$R_{I\_2} := \frac{Ru_{3\_2} \cdot 4.875 + Ru_{4\_2} \cdot 14.625 + Ru_{5\_2} \cdot 24.375 - Ru_{2\_2} \cdot 4.875 - Ru_{1\_2} \cdot 14.625}{9.75}$$

$\boxed{R_{I\_2} = 2395.66}$ kips

$$R_{H\_2} := Ru_{1\_2} + Ru_{2\_2} + Ru_{3\_2} + Ru_{4\_2} + Ru_{5\_2} - R_{I\_2}$$

$\boxed{R_{H\_2} = 490.55}$ kips

The method of joints is used to calculate the member forces. Start at node K.

By inspection, the following are zero force members and can be ignored in the model:

$F_{JK} := 0$ $F_{EK} := 0$ $F_{AF} := 0$ $F_{FG} := 0$

Note: all forces shown are in kips. "C" indicates compression and "T" indicates tension.

At node E:

$F_{EJ\_vert} := Ru_{5\_2}$ $\boxed{F_{EJ\_vert} = 706.38}$

$F_{EJ\_horiz} = Ru_{5\_2} \cdot \frac{EJ_h}{EJ_v}$ $\boxed{F_{EJ\_horiz} = 735.42}$

$F_{EJ} := \sqrt{F_{EJ\_vert}^2 + F_{EJ\_horiz}^2}$ $\boxed{F_{EJ} = 1019.71}$ C

$F_{DE} := F_{EJ\_horiz}$ $\boxed{F_{DE} = 735.42}$ T

At node J:

$F_{IJ\_horiz} := F_{EJ\_horiz}$ $\boxed{F_{IJ\_horiz} = 735.42}$

$F_{IJ\_vert} = F_{IJ\_horiz} \cdot \frac{0.802}{4.875}$ $\boxed{F_{IJ\_vert} = 120.99}$

$F_{IJ} := \sqrt{F_{IJ\_horiz}^2 + F_{IJ\_vert}^2}$ $\boxed{F_{IJ} = 745.31}$ C

$F_{DJ} := F_{EJ\_vert} - F_{IJ\_vert}$ $\boxed{F_{DJ} = 585.4}$ T

At node D:

$$F_{DI\_vert} := F_{DJ} + Ru_{4\_2}$$ $\boxed{F_{DI\_vert} = 1326.77}$

$$F_{DI\_horiz} = F_{DI\_vert} \cdot \frac{4.875}{10.167}$$ $\boxed{F_{DI\_horiz} = 636.18}$

$$F_{DI} := \sqrt{F_{DI\_vert}^{\ 2} + F_{DI\_horiz}^{\ 2}}$$ $\boxed{F_{DI} = 1471.41}$ C

$$F_{CD} := F_{DE} + F_{DI\_horiz}$$ $\boxed{F_{CD} = 1371.6}$ T

At node I:

$\boxed{R_{I\_2} = 2395.66}$

$$F_{CI\_vert} := R_{I\_2} - F_{DI\_vert} - F_{IJ\_vert}$$ $\boxed{F_{CI\_vert} = 947.9}$

$$F_{CI\_horiz} = F_{CI\_vert} \cdot \frac{4.875}{10.167}$$ $\boxed{F_{CI\_horiz} = 454.51}$

$$F_{CI} := \sqrt{F_{CI\_vert}^{\ 2} + F_{CI\_horiz}^{\ 2}}$$ $\boxed{F_{CI} = 1051.23}$ C

$$F_{HI} := F_{DI\_horiz} + F_{IJ\_horiz} - F_{CI\_horiz}$$ $\boxed{F_{HI} = 917.09}$ C

Similar calculations are performed to determine the member forces for the remainder of the model and for the load cases with one and three lanes loaded. The results are summarized in the following figures:

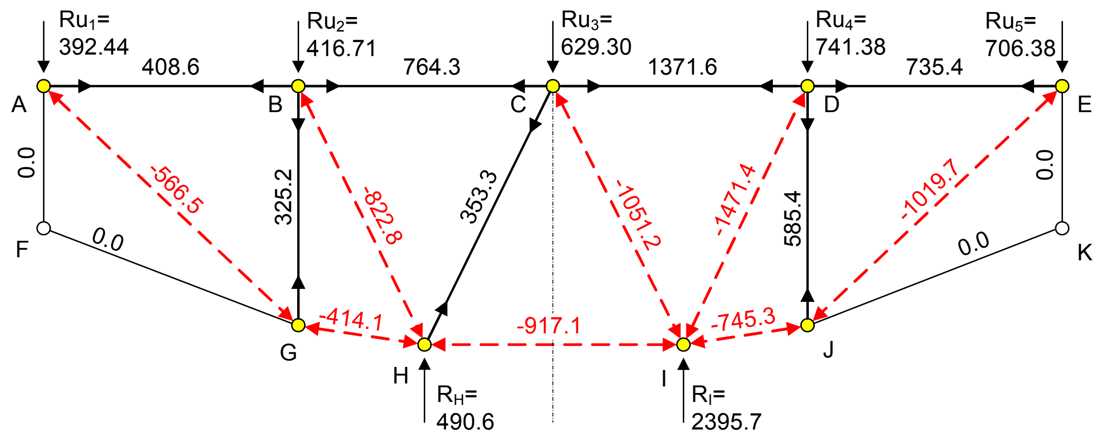

Figure E13-1.8-2
STM Member Forces (Two Lanes Loaded)

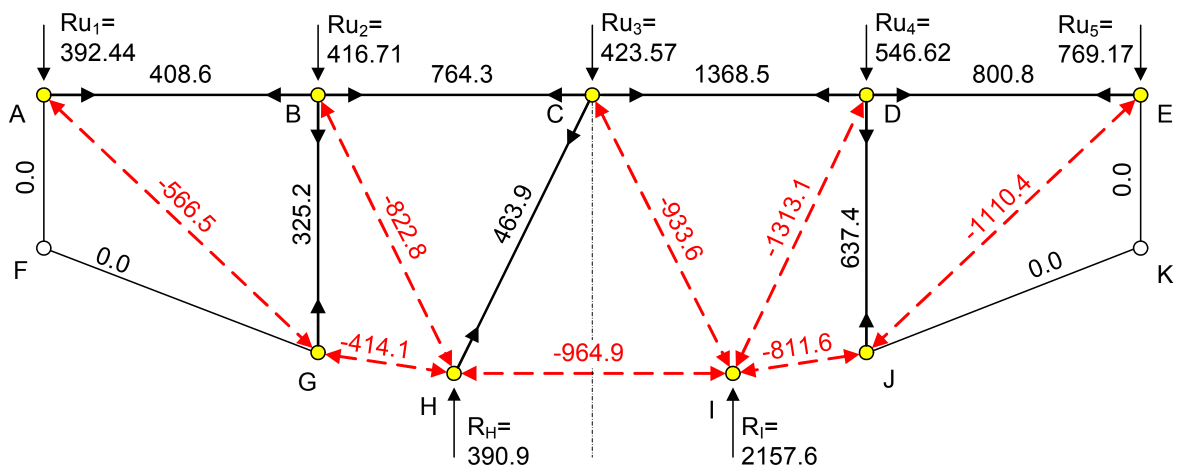

Figure E13-1.8-3
STM Member Forces (One Lane Loaded)

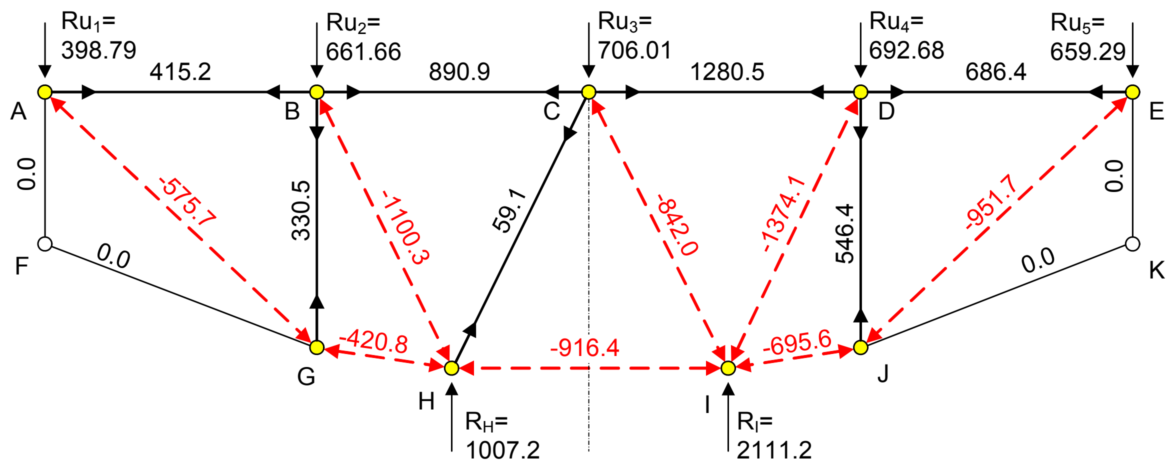

Figure E13-1.8-4

STM Member Forces (Three Lanes Loaded)

E13-1.8.2 Check the Size of the Bearings

The node conditions are defined by the connecting types of struts and ties. The following table describes the types of nodes and their corresponding limiting stresses.

Node Conditions	Description	Limiting Concrete Compressive Stress
CCC node	Strut anchored by bearing plate and strut or continuous beam support.	$0.85\Phi f'_c$
CCT node	Struct anchored by one directional tension tie.	$0.75\Phi f'_c$
CTT node	Strut anchred by tension ties in more than one direction.	$0.65\Phi f'_c$

Table E13-1.8-1

Limiting Stresses for Pier Cap Nodes

Node C: CTT
Node D: CTT
Node E: CCT
Node H: CCT
Node I: CCC

$\phi_{brg} := 0.70$

At node D the critical concrete compressive strength equals:

$0.65 \cdot \phi_{brg} \cdot f'_c = 1.59$ ksi

At node E the critical concrete compressive strength equals:

$0.75 \cdot \phi_{brg} \cdot f'_c = 1.84$ ksi

Bearing area required:

$\boxed{Ru_{4\_2} = 741.38}$

$\boxed{\gamma_{DCmax} \cdot Cap_{DC\_4} = 73.58}$

$$BrgD_2 := \frac{Ru_{4\_2} - \gamma_{DCmax} \cdot Cap_{DC\_4}}{0.65 \cdot \phi_{brg} \cdot f'_c} \qquad \boxed{BrgD_2 = 419.34} \quad in^2$$

$\boxed{Ru_{5\_1} = 769.17}$

$\boxed{\gamma_{DCmax} \cdot Cap_{DC\_5} = 43.15}$

$$BrgE_1 := \frac{Ru_{5\_1} - \gamma_{DCmax} \cdot Cap_{DC\_5}}{0.75 \cdot \phi_{brg} \cdot f'_c} \qquad \boxed{BrgE_1 = 395.11} \quad in^2$$

The area provided by the bearing plate is:

$$A_{brng} := L_{brng} \cdot W_{brng} \qquad \boxed{A_{brng} = 468} \quad in^2 \quad OK$$

E13-1.8.3 Calculate the Tension Tie Reinforcement

For the top reinforcement over the column, the required area of tension tie reinforcement, A_{st}, in Tie CD for two lanes loaded is calculated as follows:

$\boxed{Pu_{CD\_2} = 1371.6}$ kips

$\phi := 0.9$

$$Ast_{CD} := \frac{Pu_{CD\_2}}{\phi \cdot f_y} \qquad \boxed{Ast_{CD} = 25.4} \quad in^2$$

Therefore use one row of 9 No.11 bars and one row of 9 No. 10 bars spaced at 5 inches for the top reinforcement.

$As_{No11} := 1.5625$ in^2

$As_{No10} := 1.2656$ in^2

$As_{CD} := 9 \cdot As_{No11} + 9 \cdot As_{No10}$ $\boxed{As_{CD} = 25.45}$ in^2

Is $As_{CD} \geq Ast_{CD}$? $\boxed{check = \text{"OK"}}$

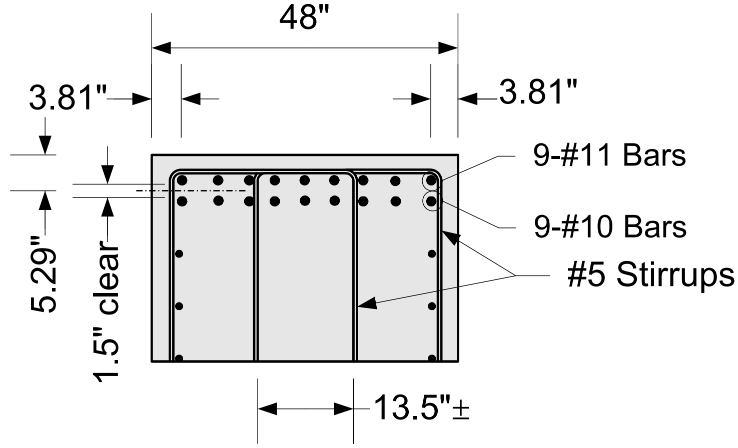

Figure E13-1.8-5
Cap Reinforcement at Tension Tie CD

Note: See **LRFD [5.10.3.1.3]** for spacing requirements between layers of rebar.

For the top reinforcement past the first interior girder, the required area of tension tie reinforcement, A_{st}, in Tie DE for two lanes loaded is calculated as follows:

$\boxed{Pu_{DE\_1} = 800.79}$ kips

$\boxed{\phi = 0.9}$

$$Ast_{DE} := \frac{Pu_{DE\_1}}{\phi \cdot f_y}$$ $\boxed{Ast_{DE} = 14.83}$ in^2

Therefore use one row of 9 No.11 bars spaced at 5 inches, and one row of 5 No.10 bars for the top reinforcement.

$As_{DE} := 9 \cdot As_{No11} + 4 \cdot As_{No10}$ $\boxed{As_{DE} = 19.12}$ in^2

Is $As_{DE} \geq Ast_{DE}$? $\boxed{check = \text{"OK"}}$

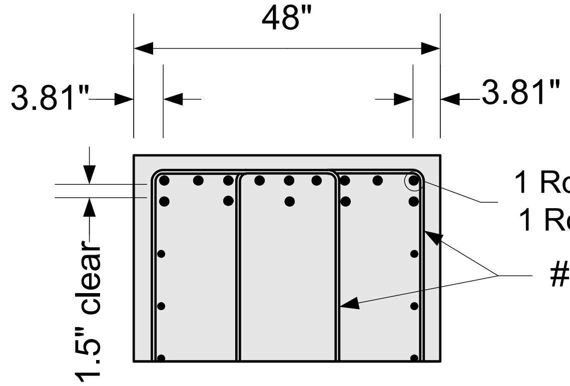

Figure E13-1.8-6
Cap Reinforcement at Tension Tie DE

E13-1.8.4 Calculate the Stirrup Reinforcement

The vertical tension ties DJ must resist a factored tension of force as shown below. The controlling force occurs with one lane loaded. This tension force will be resisted by stirrups with in the specified length of the pier cap. Note that any tension ties located directly over the column do not require stirrup design.

$\boxed{Pu_{DJ\_1} = 637.43}$ kips

$$n := \frac{Pu}{\phi \cdot A_{st} \cdot f_y}$$

Try number 5 bars, with four legs.

$As_{No5} := 0.3068$ in^2

$Ast := 4 \cdot As_{No5}$ $\boxed{Ast = 1.23}$ in^2

$$n_{DJ} := \frac{Pu_{DJ\_1}}{\phi \cdot Ast \cdot f_y} \qquad n_{DJ} = 9.62$$

$\boxed{n_{DJ} = 10}$ bars

The length over which the stirrup shall be distributed is from the face of the column to half way between girders 4 and 5.

$\boxed{S = 9.75}$ feet

$$L_{DJ} := 1.5 \cdot S - \frac{L_{col}}{2}$$ $\boxed{L_{DJ} = 6.88}$ feet

Therefore the required spacing, s, within this region is:

$s_{stirrup} := \dfrac{L_{DJ} \cdot 12}{n_{DJ}}$ $\boxed{s_{stirrup} = 8.25}$ in

$\boxed{s_{stirrup} = 8}$ in

Crack control in disturbed regions:

$\dfrac{A_{st}}{bs} \geq 0.003$

$b_v := W_{cap} \cdot 12$ $\boxed{b_v = 48}$ in

$s_{cc} := \dfrac{Ast}{0.003 \cdot b_v}$ $\boxed{s_{cc} = 8.52}$ in

$\boxed{s_{cc} = 8}$ in

$s_{stir} := \min(s_{stirrup}, s_{cc})$ $\boxed{s_{stir} = 8}$ in

$As_{DJ} := L_{DJ} \cdot Ast \cdot \dfrac{12}{8}$ $\boxed{As_{DJ} = 12.66}$ in^2

Therefore use No. 5 double-legged stirrups at 8 inch spacing in the pier cap.

E13-1.8.5 Compression Strut Capacity - Bottom Strut

After the tension tie reinforcement has been designed, the next step is to check the capacity of the compressive struts in the pier cap versus the limiting compressive stress. Strut IJ carries the highest bottom compressive force when one lane is loaded. Strut IJ is anchored by Node J, which also anchors ties DJ and strut EJ, From the geometry of the idealized internal truss, the smallest angle is between Tie DJ and Strut IJ:

$\alpha_s := \operatorname{atan}\left(\dfrac{IJ_h}{IJ_v}\right)$ $\boxed{\alpha_s = 80.66 \cdot deg}$

$\theta := 90deg - \alpha_s$ $\boxed{\theta = 9.34 \cdot deg}$

$\boxed{Pu_{IJ\_1} = -811.55}$ kips

Based on the design of the tension tie reinforcement, the tensile strain in Tie DJ is:

$\varepsilon_s := \dfrac{P_u}{A_{st} \cdot E_s}$

$E_s := 29000$ ksi

$\boxed{Pu_{DJ\_1} = 637.43}$ kips

$\boxed{L_{DJ} = 6.88}$ feet

$\boxed{s_{stir} = 8}$ inches

$$Ast_{DJ} := \frac{L_{DJ} \cdot 12}{s_{stir}} \cdot Ast \qquad \boxed{Ast_{DJ} = 12.66} \quad in^2$$

$$\varepsilon_s := \frac{Pu_{DJ\_1}}{Ast_{DJ} \cdot E_s} \qquad \boxed{\varepsilon_s = 0.00174} \quad in/in$$

Therefore, the principal strain, ε_1, can be determined **LRFD [5.6.3.3.3]**:

$$\varepsilon_1 := \varepsilon_s + \left(\varepsilon_s + 0.002\right) \cdot \cot\left(\alpha_s\right)^2 \qquad \boxed{\varepsilon_1 = 0.00184} \quad in/in$$

The limiting compressive stress, f_{cu}, in the strut can also be computed **LRFD [5.6.3.3.3]**:

$$f_{cu} = \frac{f'_c}{0.8 + 170 \cdot \varepsilon_1} \le 0.85 \cdot f'_c$$

$$f_{cu1} := \frac{f'_c}{0.8 + 170 \cdot \varepsilon_1} \qquad \boxed{f_{cu1} = 3.15} \quad ksi$$

$$f_{cu2} := 0.85 \cdot f'_c \qquad \boxed{f_{cu2} = 2.98} \quad ksi$$

$$f_{cu} := \min\left(f_{cu1}, f_{cu2}\right) \qquad \boxed{f_{cu} = 2.98} \quad ksi$$

The nominal resistance of Strut IJ is computed based on the limiting stress, f_{cu}, and the strut dimensions. The centroid of the strut was assumed to be at $\boxed{centroid_{bot} = 4.5}$ inches vertically from the bottom face. Therefore, the thickness of the strut perpendicular to the sloping bottom face is:

$$t_{IJ} := 2 \cdot centroid_{bot} \cdot \cos(\theta) \qquad \boxed{t_{IJ} = 8.88} \quad inches$$

$$w_{IJ} := W_{cap} \cdot 12 \qquad \boxed{w_{IJ} = 48} \quad inches$$

$$Acs_{IJ} := t_{IJ} \cdot w_{IJ} \qquad \boxed{Acs_{IJ} = 426.27} \quad in^2$$

$$Pn_{IJ} := f_{cu} \cdot Acs_{IJ} \qquad \boxed{Pn_{IJ} = 1268.15} \quad kips$$

$\phi c_{STM} := 0.7$

$Pr_{IJ} := \phi c_{STM} \cdot Pn_{IJ}$ $\boxed{Pr_{IJ} = 887.71}$ kips

$\boxed{\left|Pu_{IJ\_1}\right| = 811.55}$ kips

Is $Pr_{IJ} \geq Pu_{IJ\_1}$? $\boxed{check = \text{"OK"}}$

E13-1.8.6 Compression Strut Capacity - Diagonal Strut

Strut DI carries the highest diagonal compressive force when two lanes are loaded. Strut DI is anchored by Node D, which also anchors ties CD, DE and DJ, From the geometry of the idealized internal truss, the smallest angle between Tie CD and Strut DI:

$\alpha_s := \text{atan}\left(\frac{DI_v}{DI_h}\right)$ $\boxed{\alpha_s = 64.38 \cdot deg}$

$\theta := 90deg - \alpha_s$ $\boxed{\theta = 25.62 \cdot deg}$

$\boxed{Pu_{DI\_2} = -1471.41}$ kips

The tensile strain in Ties CD and DE are calculated as follows. The average of these two strains is used to check the capacity of Strut DI.

$\boxed{Pu_{CD\_2} = 1371.6}$ kips $\boxed{As_{CD} = 25.45}$ in^2

$\boxed{Pu_{DE\_2} = 735.42}$ kips $\boxed{As_{DE} = 19.12}$ in^2

$\varepsilon_{sCD\_2} := \frac{Pu_{CD\_2}}{As_{CD} \cdot E_s}$ $\boxed{\varepsilon_{sCD\_2} = 0.00186}$ $\frac{in}{in}$

$\varepsilon_{sDE\_2} := \frac{Pu_{DE\_2}}{As_{DE} \cdot E_s}$ $\boxed{\varepsilon_{sDE\_2} = 0.00133}$ $\frac{in}{in}$

$\varepsilon_{s\_ave} := \frac{\varepsilon_{sCD\_2} + \varepsilon_{sDE\_2}}{2}$ $\boxed{\varepsilon_{s\_ave} = 0.00159}$ $\frac{in}{in}$

Therefore, the principal strain, ε_1, can be determined **LRFD [5.6.3.3.3]**:

$$\varepsilon_1 := \varepsilon_{s\_ave} + (\varepsilon_{s\_ave} + 0.002)\cdot\cot(\alpha_s)^2 \qquad \boxed{\varepsilon_1 = 0.00242} \qquad \frac{in}{in}$$

The limiting compressive stress, f_{cu}, in the strut can also be computed **LRFD [5.6.3.3.3]**:

$$f_{cu} = \frac{f'_c}{0.8 + 170\cdot\varepsilon_1} \le 0.85\cdot f'_c$$

$$f_{cu1} := \frac{f'_c}{0.8 + 170\cdot\varepsilon_1} \qquad \boxed{f_{cu1} = 2.89} \qquad ksi$$

$$f_{cu2} := 0.85\cdot f'_c \qquad \boxed{f_{cu2} = 2.98} \qquad ksi$$

$$f_{cu} := \min(f_{cu1}, f_{cu2}) \qquad \boxed{f_{cu} = 2.89} \qquad ksi$$

The cross sectional dimension of Strut DI in the plane of the pier is calculated as follows. Note that for skewed bearings, the length of the bearing is the projected length along the centerline of the pier cap.

$\boxed{L_{brng} = 26}$ inches

$\boxed{W_{brng} = 18}$ inches

$\boxed{centroid_{top} = 5.5}$ inches

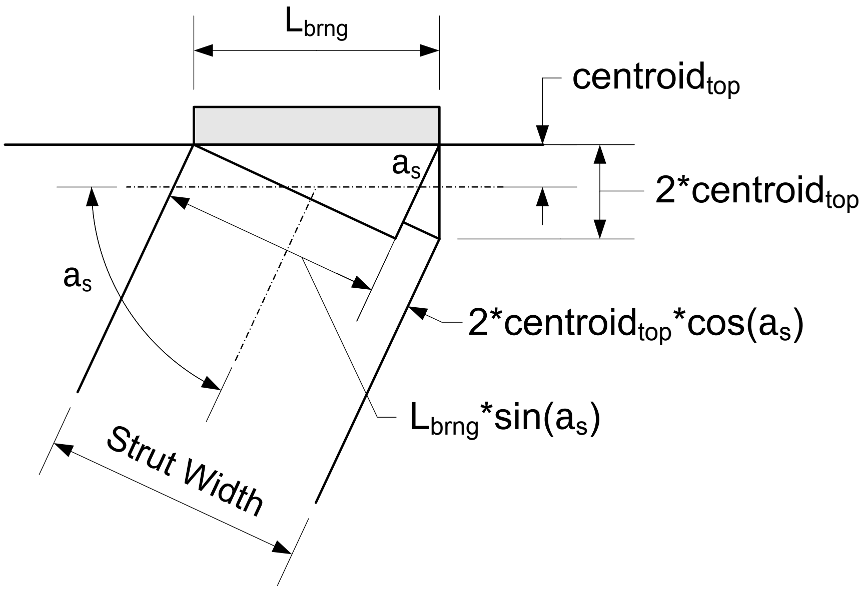

Figure E13-1.8-7
Compression Strut Width

$t_{DI} := L_{brng} \cdot \sin(\alpha_s) + 2 \cdot centroid_{top} \cdot \cos(\alpha_s)$ $\boxed{t_{DI} = 28.2}$ in

The effective compression strut width around each stirrup is:

$d_{bar11} := 1.410$ inches

$w_{ef} := 2 \cdot 6 \cdot d_{bar11}$ $\boxed{w_{ef} = 16.92}$ in

The effective spacing between the 4 legs of the stirrups is 13.5 inches, which is less than the value calculated above. Therefore, the entire cap width can be used for the effective strut width.

$w_{DI} := W_{cap} \cdot 12$ $\boxed{w_{DI} = 48}$ in

The nominal resistance of Strut DI is computed based on the limiting stress, f_{cu}, and the strut dimensions.

$Acs_{DI} := t_{DI} \cdot w_{DI}$ $\boxed{Acs_{DI} = 1353.61}$ in^2

$Pn_{DI} := f_{cu} \cdot Acs_{DI}$ $\boxed{Pn_{DI} = 3911.99}$ kips

$\boxed{\phi c_{STM} = 0.7}$

$Pr_{DI} := \phi c_{STM} \cdot Pn_{DI}$ $\boxed{Pr_{DI} = 2738.4}$ kips

$\boxed{\left|Pu_{DI\_2}\right| = 1471.41}$ kips

Is $Pr_{DI} \geq \left|Pu_{DI\_2}\right|$? $\boxed{check = \text{"OK"}}$

E13-1.8.7 Check the Anchorage of the Tension Ties

12 No. 11 longitudinal bars along the top of the pier cap must be developed at the inner edge of the bearing at Node E (the edge furthest from the end of the member). Based on **Figure E13-1.8-8**, the embedment length that is available to develop the bar beyond the edge of the bearing is:

L_{devel} = (distance from end to Node) + (bearing block width/2) - (cover)

$\boxed{L_{cap} = 46.5}$ feet

$\boxed{S = 9.75}$ feet

$\boxed{L_{brng} = 26}$ inches

$\boxed{Cover_{cp} = 2.5}$ inches

$$L_{devel} := \frac{L_{cap} - S\cdot(ng-1)}{2}\cdot 12 + \frac{L_{brng}}{2} - Cover_{cp} \qquad \boxed{L_{devel} = 55.5} \quad \text{in}$$

The basic development length for straight No. 11 and No. 10 bars with spacing less than 6", As(provided)/As(required) < 2, uncoated top bar, per **Wis Bridge Manual Table 9.9-1** is:

$\boxed{L_{d11} := 9.5}$ ft $\qquad \boxed{L_{d11}\cdot 12 = 114}$ in

$\boxed{L_{d10} := 7.75}$ ft $\qquad \boxed{L_{d10}\cdot 12 = 93}$ in

Therefore there is not sufficient development length for straight bars. Check the hook development length. The base hook development length for 90º hooked No.11 and #10 bars per **LRFD [5.11.2.4]** is:

$$L_{hb11} := \frac{38.0\cdot d_{bar11}}{\sqrt{f'_c}} \qquad \boxed{L_{hb11} = 28.64} \quad \text{in}$$

$$L_{hb10} := \frac{38.0\cdot d_{bar10}}{\sqrt{f'_c}} \qquad \boxed{L_{hb10} = 25.8} \quad \text{in}$$

The length available is greater than the base hook development length, therefore the reduction factors do not need to be considered. Hook both the top 9 bars and the bottom layer 5 bars. The remaining 4 bottom layer bars can be terminated 7.75 feet from the inside edge of the bearings at girders 2 and 4.

In addition, the tension ties must be spread out sufficiently in the effective anchorage area. The centroid of the tension ties is $\boxed{centroid_{top} = 5.5}$ inches below the top of the pier cap. Therefore, the effective depth of the anchorage area is 11 inches. The nodal zone stress to anchor the tension tie is:

$\boxed{Pu_{DE\_1} = 800.79}$ kips

$\boxed{centroid_{top} = 5.5}$ inches

$$f_c := \frac{Pu_{DE\_1}}{2\cdot centroid_{top}\cdot W_{cap}\cdot 12} \qquad \boxed{f_c = 1.52} \quad \text{ksi}$$

This nodal region anchors a one direction tension tie, and Node E is classified as a CCT node. The limiting nodal zone stress presented in **Table 13-1.8-1** is:

$\boxed{0.75\cdot\phi\cdot f'_c = 2.36}$ ksi

Is $0.75\cdot\phi\cdot f'_c \geq f_c$? $\qquad \boxed{check = \text{"OK"}}$

Therefore, the requirement for the nodal zone stress limit in the anchorage area is satisfied.

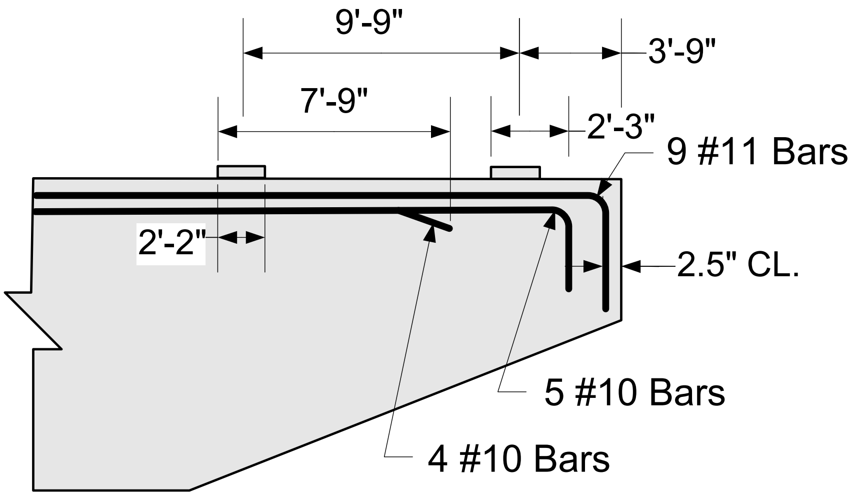

Figure E13-1.8-8
Anchorage of Tension Tie

E13-1.8.8 Provide Crack Control Reinforcement

In the disturbed regions, the minimum ratio of reinforcement to the gross concrete area is 0.003 in each direction, and the spacing of the bars in these grids must not exceed 12 inches, **LRFD [5.6.3.6]**. Therefore the required crack control reinforcement within a 1 foot section is:

$As_{crack} := 0.003 \cdot (12) \cdot W_{cap} \cdot 12$ $\boxed{As_{crack} = 1.73}$ in^2

Use 4 - No. 7 horizontal bars at 12 inch spacing in the vertical direction

$As_{No7} := 0.6013$ $\boxed{4 \cdot As_{No7} = 2.41}$ in^2

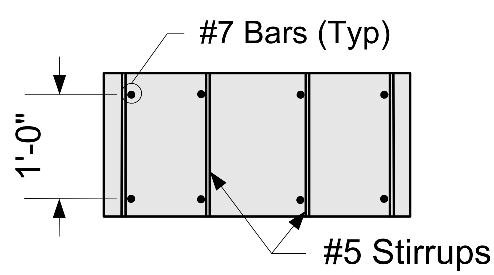

Figure E13-1.8-9
Crack Control Reinforcement - Option 1

OR If we assume 6-inch vertical spacing

$As_{crack} := 0.003 \cdot (6) \cdot W_{cap} \cdot 12$ $\boxed{As_{crack} = 0.86}$ in^2

$2 \cdot As_{No7} = 1.2$ in^2

Is $2 \cdot As_{No7} \geq As_{crack}$? $check = \text{"OK"}$

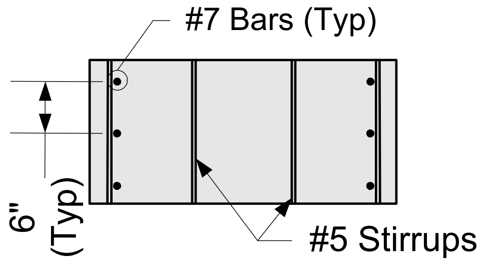

Figure E13-1.8-10
Crack Control Reinforcement - Option 2

This 6-inch spacing for the number 7 temperature and shrinkage reinforcement is also used along the bottom of the cap.

The stirrups are spaced at, $s_{stir} = 8$ inches. Therefore the required crack control reinforcement within this spacing is:

$As_{crack2} := 0.003 \cdot (s_{stir}) \cdot W_{cap} \cdot 12$ $As_{crack2} = 1.15$ in^2

4 legs of No.5 stirrups at $s_{stir} = 8$ inch spacing in the horizontal direction

$4 \cdot As_{No5} = 1.23$ in^2

Is $4 \cdot As_{No5} \geq As_{crack2}$? $check = \text{"OK"}$

E13-1.8.9 Summary of Cap Reinforcement

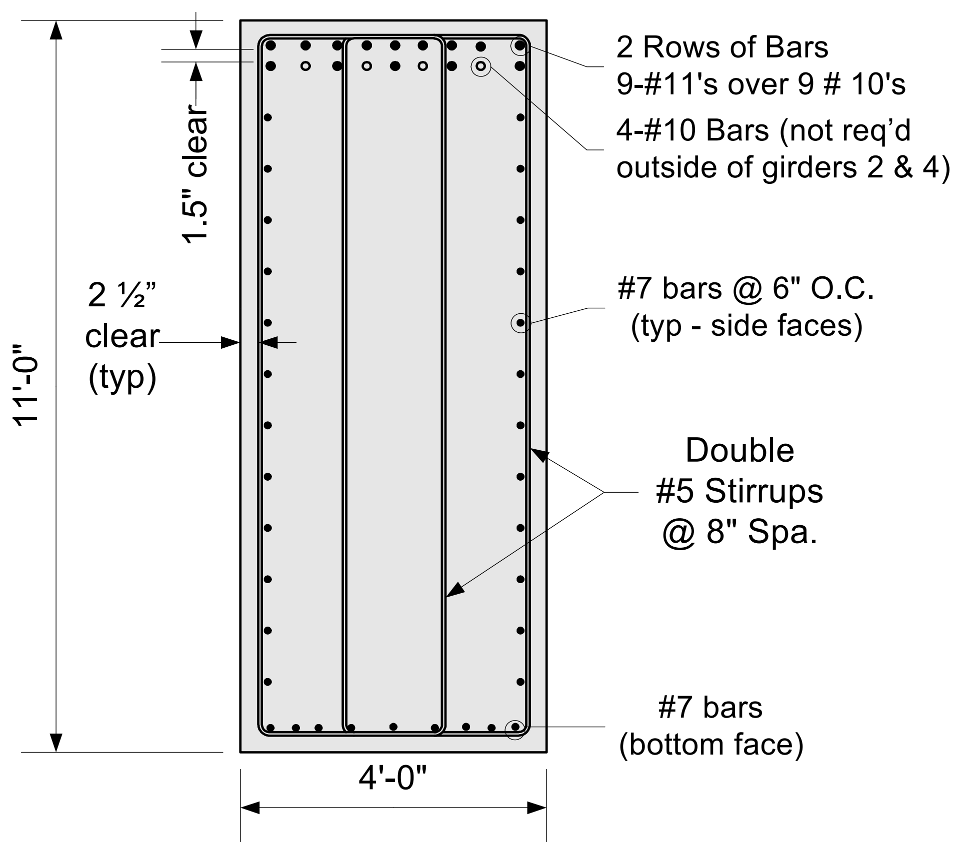

Figure E13-1.8-11
Pier Cap Design Summary

E13-1.9 Design Pier Column

As stated in E13-1.7, the critical section in the pier column is where the column meets the footing, or at the column base. The governing force effects and their corresponding limit states were determined to be:

Strength V

$\boxed{Ax_{colStrV} = 2054.87}$ kips

$\boxed{MuT_{colStrV} = 8789.59}$ kip-ft

$\boxed{MuL_{colStrV} = 2333.6}$ kip-ft

Strength III

$\boxed{VuT_{col} = 76.45}$ kips

Strength V

$\boxed{VuL_{col} = 105.37}$ kips

A preliminary estimate of the required section size and reinforcement is shown in Figure E13-1.9-1.

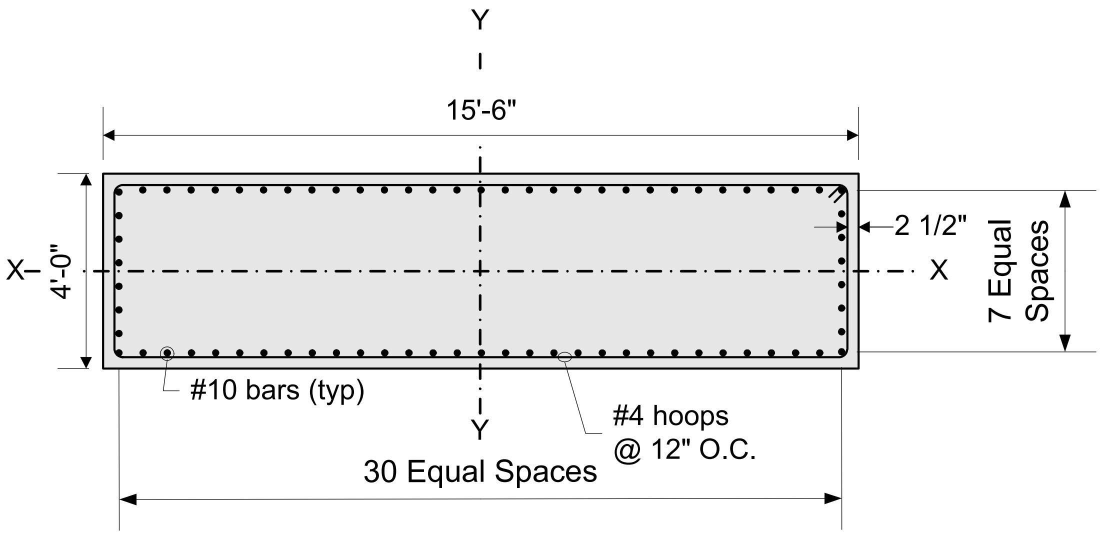

Figure E13-1.9-1
Preliminary Pier Column Design

E13-1.9.1 Design for Axial Load and Biaxial Bending (Strength V):

The preliminary column reinforcing is show in Figure E13-1.9-1 and corresponds to #10 bars equally spaced around the column perimeter. **LRFD [5.7.4.2]** prescribes limits (both maximum and minimum) on the amount of reinforcing steel in a column. These checks are performed on the preliminary column as follows:

$Num\_bars := 74$ $bar\_area10 := 1.27$ in² $bar\_dia10 := 1.27$ in

$A_{s\_col} := (Num\_bars) \cdot (bar\_area10)$ $\boxed{A_{s\_col} = 93.98}$ in²

$A_{g\_col} := (W_{col}) \cdot (L_{col}) \cdot 12^2$ $\boxed{A_{g\_col} = 8928}$ in²

$\frac{A_{s\_col}}{A_{g\_col}} = 0.0105$ $0.0105 \le 0.08$ (max. reinf. check) OK

$\frac{0.135 \cdot f'_c}{f_y} = 0.008$ (but need not be greater than 0.015) $0.0105 \ge 0.008$ (min. reinf. check) OK

The column slenderness ratio (Kl_u/r) about each axis of the column is computed below in order to assess slenderness effects. Note that the Specifications only permit the following approximate evaluation of slenderness effects when the slenderness ratio is below 100.

For this pier, the unbraced lengths (l_{ux}, l_{uy}) used in computing the slenderness ratio about each axis is the full pier height. This is the height from the top of the footing to the top of the pier cap (26 feet). The effective length factor in the longitudinal direction, K_x, is taken equal to 2.1. This assumes that the superstructure has no effect on restraining the pier from buckling. In essence, the pier is considered a free-standing cantilever in the longitudinal direction. The effective length factor in the transverse direction, K_y, is taken to equal 1.0.

The radius of gyration (r) about each axis can then be computed as follows:

$$I_{xx} := \frac{(L_{col} \cdot 12) \cdot (W_{col} \cdot 12)^3}{12} \qquad \boxed{I_{xx} = 1714176} \quad in^4$$

$$I_{yy} := \frac{(W_{col} \cdot 12) \cdot (L_{col} \cdot 12)^3}{12} \qquad \boxed{I_{yy} = 25739424} \quad in^4$$

$$r_{xx} := \sqrt{\frac{I_{xx}}{A_{g\_col}}} \qquad \boxed{r_{xx} = 13.86} \quad in$$

$$r_{yy} := \sqrt{\frac{I_{yy}}{A_{g\_col}}} \qquad \boxed{r_{yy} = 53.69} \quad in$$

The slenderness ratio for each axis now follows:

$$K_x := 2.1$$

$$K_y := 1.0$$

$$L_u := (H_{col} + H_{cap}) \cdot 12 \qquad \boxed{L_u = 312} \quad in$$

$$\boxed{\frac{K_x \cdot L_u}{r_{xx}} = 47.28} \qquad 47.28 < 100 \quad \text{OK}$$

$$\boxed{\frac{K_y \cdot L_u}{r_{yy}} = 5.81} \qquad 5.81 < 100 \quad \text{OK}$$

LRFD [5.7.4.3] permits the slenderness effects to be ignored when the slenderness ratio is less than 22 for members not braced against side sway. It is assumed in this example that the pier is not braced against side sway in either its longitudinal or transverse directions. Therefore, slenderness will be considered for the pier longitudinal direction only (i.e., about the "X-X" axis).

In computing the amplification factor that is applied to the longitudinal moment, which is the enc result of the slenderness effect, the column stiffness (EI) about the "X-X" axis must be defined.

In doing so, the ratio of the maximum factored moment due to permanent load to the maximum factored moment due to total load must be identified (β_d).

From Design Step E13-1.7, it can be seen that the force effects contributing to the longitudinal moment are the live load braking force, the temperature force and wind on the structure and live load. None of these are permanent or long-term loads. Therefore, β_d is taken equal to zero for this design.

$$\beta_d := 0$$

$$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$$ **LRFD [C5.4.2.7]** $\boxed{E_c = 3587}$ ksi

$\boxed{E_s = 29000.00}$ ksi

$\boxed{I_{xx} = 1714176}$ in^4

I_s = Moment of Inertia of longitudinal steel about the centroidal axis (in^4)

$$I_s := \frac{\pi \cdot bar\_dia10^4}{64} \cdot (Num\_bars) + 2 \cdot 31 \cdot (bar\_area10) \cdot 20.37^2 \ldots$$
$$+ 4 \cdot (bar\_area10) \cdot 14.55^2 + 4 \cdot (bar\_area10) \cdot 8.73^2 + 4 \cdot (bar\_area10) \cdot 2.91^2$$

$\boxed{I_s = 34187}$ in^4

The column stiffness is taken as the greater of the following two calculations:

$$EI_1 := \frac{\frac{E_c \cdot I_{xx}}{5} + E_s \cdot I_s}{1 + \beta_d}$$ $\boxed{EI_1 = 2.22 \times 10^9}$ k-in²

$$EI_2 := \frac{\frac{E_c \cdot I_{xx}}{2.5}}{1 + \beta_d}$$ $\boxed{EI_2 = 2.46 \times 10^9}$ k-in²

$$EI := \max(EI_1, EI_2)$$ $\boxed{EI = 2.46 \times 10^9}$ k-in²

The final parameter necessary for the calculation of the amplification factor is the phi-factor for compression. This value is defined as follows:

$$\phi_{axial} := 0.75$$

It is worth noting at this point that when axial load is present in addition to flexure, **LRFD [5.5.4.2.1]** permits the value of phi to be increased linearly to the value for flexure (0.90) as the section changes from compression controlled to tension controlled as defined in **LRFD [5.7.2.1]**. However, certain equations in the Specification still require the use of the phi factor for axial compression (0.75) even when the increase just described is permitted. Therefore, for the sake of clarity in this example, if phi may be increased it will be labeled separately from ϕ_{axial} identified above.

$As_{col} := 2.53$ in² per foot, based on #10 bars at 6-inch spacing

$b := 12$ inches $\alpha_1 := 0.85$ (for $f'_c < 10.0$ ksi) **LRFD [5.7.2.2]**

$$a := \frac{As_{col} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b} \qquad \boxed{a = 4.25} \text{ inches}$$

$\beta_1 := 0.85$

$$c := \frac{a}{\beta_1} \qquad \boxed{c = 5.00} \text{ inches}$$

$$d_t := W_{col} \cdot 12 - Cover_{co} - 0.5 - \frac{bar\_dia10}{2} \qquad \boxed{d_t = 44.37} \text{ inches}$$

$\varepsilon_c := 0.002$ Upper strain limit for compression controlled sections, $f_y = 60$ ksi **LRFD [Table C5.7.2-1]**

$\varepsilon_t := 0.005$ Lower strain limit for tension controlled sections, for $f_y = 60$ ksi

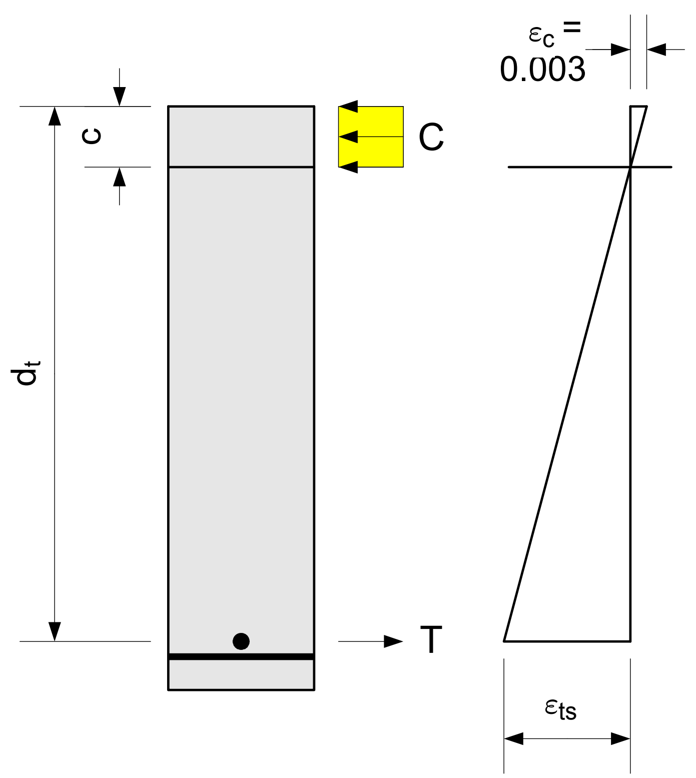

Figure E13-1.9-2
Strain Limit Tension Control Check

$$\varepsilon_{ts} := \frac{\varepsilon_c}{c} \cdot (d_t - c) \qquad \boxed{\varepsilon_{ts} = 0.016} \qquad > \varepsilon_t = 0.005$$

Therefore, the section is tension controlled and phi shall be equal to 0.9.

$\phi_t := 0.9$

The longitudinal moment magnification factor will now be calculated as follows:

$$P_e := \frac{\pi^2 \cdot EI}{(K_x \cdot L_u)^2} \qquad \boxed{P_e = 56539.53} \text{ kips}$$

$$\delta_s := \frac{1}{1 - \left(\frac{Ax_{colStrV}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_s = 1.04}$$

The final design forces at the base of the column for the Strength I limit state will be redefined as follows:

$$P_{u\_col} := Ax_{colStrV} \qquad \boxed{P_{u\_col} = 2054.87} \text{ kips}$$

$$M_{ux} := MuL_{colStrV} \cdot \delta_s \qquad \boxed{M_{ux} = 2431.8} \text{ kip-ft}$$

$$M_{uy} := MuT_{colStrV} \qquad \boxed{M_{uy} = 8789.59} \text{ kip-ft}$$

The assessment of the resistance of a compression member with biaxial flexure for strength limit states is dependent upon the magnitude of the factored axial load. This value determines which of two equations provided by the Specification are used.

If the factored axial load is less than ten percent of the gross concrete strength multiplied by the phi-factor for compression members (ϕ_{axial}), then the Specifications require that a linear interaction equation for only the moments is satisfied (**LRFD [Equation 5.7.4.5-3]**). Otherwise, an axial load resistance (P_{rxy}) is computed based on the reciprocal load method (**LRFD [Equation 5.7.4.5-1]**). In this method, axial resistances of the column are computed (using f_{Low_axial} if applicable) with each moment acting separately (i.e., P_{rx} with M_{ux}, P_{ry} with M_{uy}). These are used along with the theoretical maximum possible axial resistance (P_o multiplied by ϕ_{axial}) to obtain the factored axial resistance of the biaxially loaded column.

Regardless of which of the two equations mentioned in the above paragraph controls, commercially available software is generally used to obtain the moment and axial load resistances.

For this pier design, the procedure as discussed above is carried out as follows:

$$\boxed{0.10 \cdot \phi_{axial} \cdot f'_c \cdot A_{g\_col} = 2343.6} \text{ kips}$$

$$\boxed{P_{u\_col} = 2054.87} \text{ kips} \qquad P_{u\_col} < 2343.6K$$

Therefore, **LRFD [Equation 5.7.4.5-3]** will be used.

$\boxed{M_{ux} = 2431.8}$ kip-ft $\boxed{M_{uy} = 8789.59}$ kip-ft

The resultant moment equals:

$Mu := \sqrt{M_{ux}^{2} + M_{uy}^{2}}$ $\boxed{Mu = 9119.79}$ kip-ft

$M_r := 24052.3$ kip-ft

$\boxed{\frac{Mu}{M_r} = 0.38}$ $0.38 \le 1.0$ OK

The factored flexural resistances shown above, M_r, was obtained by the use of commercial software. This value is the resultant flexural capacity assuming that no axial load is present. Consistent with this, the phi-factor for flexure (0.90) was used in obtaining the factored resistance from the factored nominal strength.

Although the column has a fairly large excess flexural capacity, a more optimal design will not be pursued per the discussion following the column shear check.

E13-1.9.2 Design for Shear (Strength III and Strength V)

The maximum factored transverse and longitudinal shear forces were derived in E13-1.7 and are as follows:

$\boxed{VuT_{col} = 76.45}$ kips (Strength III)

$\boxed{VuL_{col} = 105.37}$ kips (Strength V)

These maximum shear forces do not act concurrently. Although a factored longitudinal shear force is present in Strength III and a factored transverse shear force is present in Strength V, they both are small relative to their concurrent factored shear. Therefore, separate shear designs can be carried out for the longitudinal and transverse directions using only the maximum shear force in that direction.

For the pier column of this example, the maximum factored shear in either direction is less than one-half of the factored resistance of the concrete. Therefore, shear reinforcement is not required. This is demonstrated for the longitudinal direction as follows:

$b_v := L_{col} \cdot 12$ $\boxed{b_v = 186}$ in

$h := W_{col} \cdot 12$ $\boxed{h = 48}$ in

Conservatively, d_v may be calculated as shown below, **LRFD [5.8.2.9]**.

$\boxed{d_v := (0.72) \cdot (h)}$ $\boxed{d_v = 34.56}$ in

The above calculation for d_v is simple to use for columns and generally results in a conservative estimate of the shear capacity.

$\beta := 2.0$ $\theta := 45deg$

The nominal concrete shear strength is:

$V_c := 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_c = 760.04}$ kips

The nominal shear strength of the column is the lesser of the following two values:

$V_{n1} := V_c$ $\boxed{V_{n1} = 760.04}$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v$ $\boxed{V_{n2} = 5624.64}$ kips

$V_n := \min(V_{n1}, V_{n2})$ $\boxed{V_n = 760.04}$ kips

The factored shear resistance is:

$\phi_v := 0.90$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 684.04}$ kips

$\boxed{\frac{V_r}{2} = 342.02}$ kips

$\boxed{VuL_{col} = 105.37}$ kips $\frac{V_r}{2} > VuL_{col}$

$\boxed{check = \text{"OK"}}$

It has just been demonstrated that transverse steel is not required to resist the applied factored shear forces. However, transverse confinement steel in the form of hoops, ties or spirals is required for compression members. In general, the transverse steel requirements for shear and confinement must both be satisfied per the Specifications.

It is worth noting that although the preceding design checks for shear and flexure show the column to be over designed, a more optimal column size will not be pursued. The reason for this is twofold: First, in this design example, the requirements of the pier cap dictate the column dimensions (a reduction in the column width will increase the moment in the pier cap). Secondly, a short, squat column such as the column in this design example generally has a relatively large excess capacity even when only minimally reinforced.

E13-1.9.3 Transfer of Force at Base of Column

The provisions for the transfer of forces and moments from the column to the footing are new to the AASHTO LRFD Specifications. In general, standard engineering practice for bridge piers automatically satisfies most, if not all, of these requirements.

In this design example, and consistent with standard engineering practice, all steel reinforcing bars in the column extend into, and are developed, in the footing (see Figure E13-1.12-1).

This automatically satisfies the following requirements for reinforcement across the interface of the column and footing: A minimum reinforcement area of 0.5 percent of the gross area of the supported member, a minimum of four bars, and any tensile force must be resisted by the reinforcement. Additionally, with all of the column reinforcement extended into the footing, along with the fact that the column and footing have the same compressive strength, a bearing check at the base of the column and the top of the footing is not applicable.

In addition to the above, the Specifications require that the transfer of lateral forces from the pier to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. With the standard detailing practices for bridge piers previously mentioned (i.e., all column reinforcement extended and developed in the footing), along with identical design compressive strengths for the column and footing, this requirement is generally satisfied. However, for the sake of completeness, this check will be carried out as follows:

$A_{cv} := A_{g\_col}$ Area of concrete engaged in shear transfer. $\boxed{A_{cv} = 8928}$ in^2

$A_{vf} := A_{s\_col}$ Area of shear reinforcement crossing the shear plane. $\boxed{A_{vf} = 93.98}$ in^2

For concrete placed against a clean concrete surface, not intentionally roughened, the following values are obtained from **LRFD [5.8.4.3]**.

$c_{cv} := 0.075$ Cohesion factor, ksi

$\mu := 0.60$ Friction factor

$K_1 := 0.2$

$K_2 := 0.8$

The nominal shear-friction capacity is the smallest of the following three equations (conservatively ignore permanent axial compression):

$V_{nsf1} := c_{cv} \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y$ $\boxed{V_{nsf1} = 4052.88}$ kips

$V_{nsf2} := K_1 \cdot f'_c \cdot A_{cv}$ $\boxed{V_{nsf2} = 6249.6}$ kips

$V_{nsf3} := K_2 \cdot A_{cv}$ $\boxed{V_{nsf3} = 7142.4}$ kips

Define the nominal shear-friction capacity as follows:

$V_{nsf} := \min\left(V_{nsf1}, V_{nsf2}, V_{nsf3}\right)$ $\boxed{V_{nsf} = 4052.88}$ kips

The maximum applied shear was previously identified from the Strength V limit state:

$\boxed{VuL_{col} = 105.37}$ kips

It then follows:

$\boxed{\phi_V = 0.9}$

$\boxed{\phi_V \cdot (V_{nsf}) = 3647.59}$ kips

$\phi_V \cdot (V_{nsf}) \geq VuL_{col}$

$\boxed{\text{check} = \text{"OK"}}$

As can be seen, a large excess capacity exists for this check. This is partially due to the fact that the column itself is over designed in general (this was discussed previously). However, the horizontal forces generally encountered with common bridges are typically small relative to the shear-friction capacity of the column (assuming all reinforcing bars are extended into the footing). In addition, the presence of a shear-key, along with the permanent axial compression from the bridge dead load, further increase the shear-friction capacity at the column/footing interface beyond that shown above.

E13-1.10 Design Pier Piles

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The HP12x53 pile layout used for this pier foundation is shown in Figure E13-1.10-1.

Based on the given pile layout, the controlling limit states for the pile design were given in E13-1.7.3.

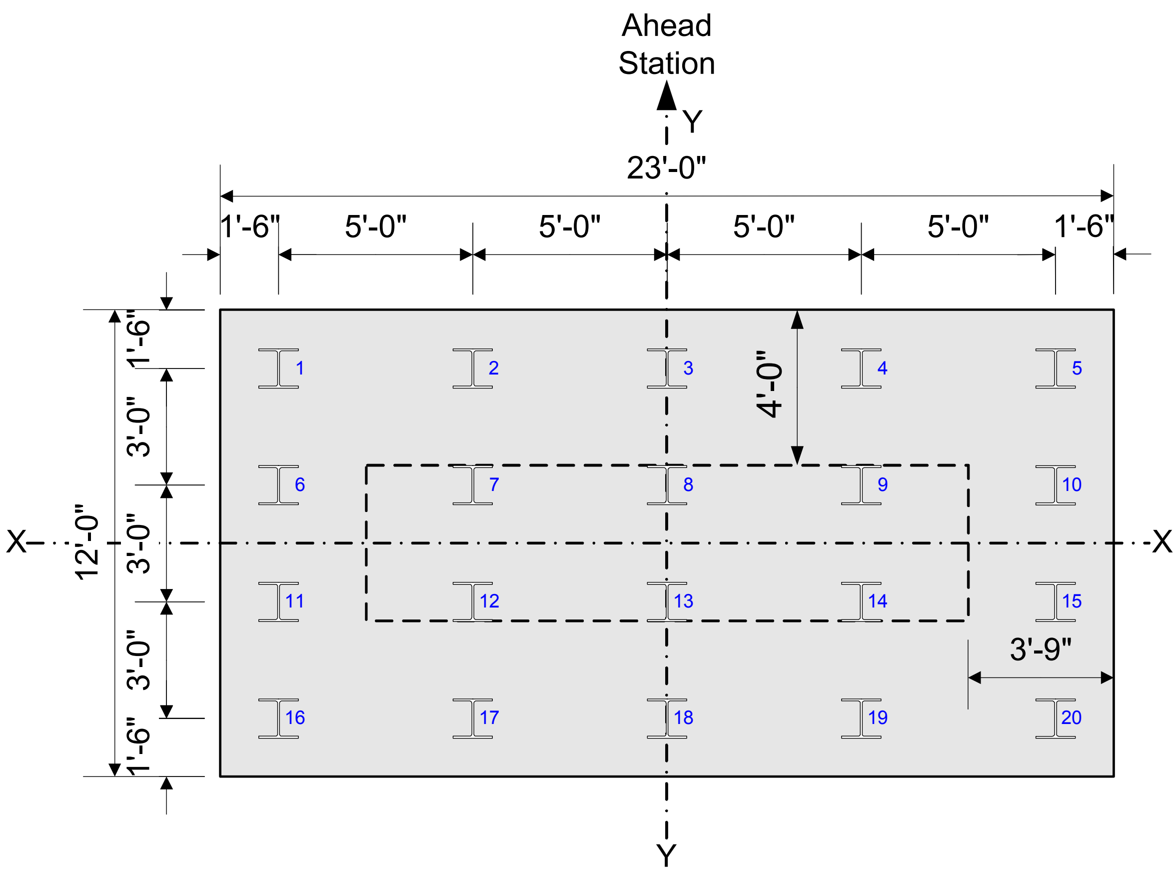

Figure E13-1.10-1

Pier Pile Layout

$N_p := 20$ Number of piles

$$S_{xx} := \frac{10 \cdot 4.5^2 + 10 \cdot 1.5^2}{4.5} \qquad \boxed{S_{xx} = 50} \quad ft^3$$

$$S_{yy} := \frac{8 \cdot 10^2 + 8 \cdot 5^2}{10} \qquad \boxed{S_{yy} = 100} \quad ft^3$$

Maximum pile reaction:

$\boxed{\phi_t = 0.9}$

$\boxed{P_e = 56539.53}$ kips (from column design)

$\boxed{Pu2_{pile\_Str1} = 3179.17}$ kips

$\boxed{MuT2_{pile\_Str1} = 7836.85}$ kip-ft

$\boxed{MuL2_{pile\_Str1} = 1856.29}$ kip-ft

$$\delta_{pile\_StrI} := \frac{1}{1 - \left(\frac{Pu2_{pile\_Str1}}{\phi_t \cdot P_e}\right)}$$

$\boxed{\delta_{pile\_StrI} = 1.07}$

$$Pu_p := \frac{Pu2_{pile\_Str1}}{N_p} + \frac{MuT2_{pile\_Str1}}{S_{yy}} + \frac{MuL2_{pile\_Str1} \cdot \delta_{pile\_StrI}}{S_{xx}}$$

$\boxed{Pu_p = 276.93}$ kips

$$Pu_{p\_tons} := \frac{Pu_p}{2}$$

$\boxed{Pu_{p\_tons} = 138.46}$ tons

From Wis Bridge Manual, Section 11.3.1.17.6, the vertical pile resistance of HP12x53 pile is :

$\boxed{Pr_{12x53} = 110}$ tons $\boxed{check = \text{"No Good"}}$

$\boxed{Pr_{12x53\_PDA} = 143}$ tons $\boxed{check = \text{"OK"}}$

Note: PDA with CAPWAP is typically used when it is more economical than modified Gates. This example uses PDA with CAPWAP only to illustrate that vertical pile reactions are satisfied and to minimize example changes due to revised pile values. The original example problem was based on higher pile values than the current values shown in Chapter 11,Table 11.3-5.

Minimum pile reaction (Strength V):

$\boxed{Pu_{pile\_StrV} = 2134.91}$ kips

$\boxed{MuT_{pile\_StrV} = 7670.61}$ kip-ft

$\boxed{MuL_{pile\_StrV} = 2333.6}$ kip-ft

$$\delta_{pile\_StrV} := \frac{1}{1 - \left(\frac{Pu_{pile\_StrV}}{\phi_t \cdot P_e}\right)}$$

$\boxed{\delta_{pile\_StrV} = 1.04}$

$$Pu_{min\_p} := \frac{Pu_{pile\_StrV}}{N_p} - \frac{MuT_{pile\_StrV}}{S_{yy}} - \frac{MuL_{pile\_StrV} \cdot \delta_{pile\_StrV}}{S_{xx}}$$

$\boxed{Pu_{min\_p} = -18.68}$ kips

Capacity for pile uplift is site dependant. Consult with the geotechnical engineer for allowable values.

The horizontal pile resistance of HP12x53 pile from the soils report is :

$Hr_{12x53} := 14$ kips/pile

Pile dimensions in the transverse (xx) and longitudinal (yy) directions:

$B_{xx} := 12.05$ inches

$B_{yy} := 11.78$ inches

Pile spacing in the transverse and longitudinal directions:

$Spa_{xx} := 5.0$ feet $\boxed{\dfrac{Spa_{xx}}{\dfrac{B_{xx}}{12}} = 4.98}$ $\boxed{\text{Say: 5B}}$

$Spa_{yy} := 3.0$ feet $\boxed{\dfrac{Spa_{yy}}{\dfrac{B_{yy}}{12}} = 3.06}$ $\boxed{\text{Say: 3B}}$

Use the pile multipliers from **LRFD [T-10.7.2.4-1]** to calculate the group resistance of the piles in each direction.

$Hr_{xx} := Hr_{12x53} \cdot 4 \cdot (1.0 + 0.85 + 0.70 \cdot 3)$

$\boxed{Hr_{xx} = 221.2}$ kips

$\boxed{HuT_{pileStrIII} = 76.45}$ kips

$Hr_{xx} \geq HuT_{pileStrIII}$

$\boxed{\text{check = "OK"}}$

$Hr_{yy} := Hr_{12x53} \cdot 5 \cdot (0.7 + 0.5 + 0.35 \cdot 2)$

$\boxed{Hr_{yy} = 133}$ kips

$\boxed{HuL_{pileStrV} = 105.37}$ kips

$Hr_{yy} \geq HuL_{pileStrV}$

$\boxed{\text{check = "OK"}}$

E13-1.11 - Design Pier Footing

In E13-1.7, the Strength I limit states was identified as the governing limit state for the design of the pier footing.

Listed below are the Strength I footing loads for one, two and three lanes loaded:

$\boxed{Pu1_{ftgStr1} = 2643.74}$	kips	$\boxed{Pu2_{ftgStr1} = 2928.7}$	kips
$\boxed{MuT1_{ftgStr1} = 7267.81}$	kip-ft	$\boxed{MuT2_{ftgStr1} = 7836.85}$	kip-ft
$\boxed{MuL1_{ftgStr1} = 1187.7}$	kip-ft	$\boxed{MuL2_{ftgStr1} = 1856.29}$	kip-ft

$\boxed{Pu3_{ftgStr1} = 3124.66}$	kips
$\boxed{MuT3_{ftgStr1} = 4541.55}$	kip-ft
$\boxed{MuL3_{ftgStr1} = 2315.94}$	kip-ft

The longitudinal moment given above must be magnified to account for slenderness of the column (see E13-1.9). The computed magnification factor and final factored forces are:

$$\delta_{s1\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu1_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s1\_ftgStr1} = 1.05}$$

$$\delta_{s2\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu2_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s2\_ftgStr1} = 1.06}$$

$$\delta_{s3\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu3_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s3\_ftgStr1} = 1.07}$$

$$MuL1_{ftgStr1\delta} := \delta_{s1\_ftgStr1} \cdot MuL1_{ftgStr1} \qquad \boxed{MuL1_{ftgStr1\delta} = 1252.79} \text{ kip-ft}$$

$$MuL2_{ftgStr1\delta} := \delta_{s2\_ftgStr1} \cdot MuL2_{ftgStr1} \qquad \boxed{MuL2_{ftgStr1\delta} = 1969.65} \text{ kip-ft}$$

$$MuL3_{ftgStr1\delta} := \delta_{s3\_ftgStr1} \cdot MuL3_{ftgStr1} \qquad \boxed{MuL3_{ftgStr1\delta} = 2467.46} \text{ kip-ft}$$

The calculations for the Strength I pile loads on the footing are calculated below for one, two and three lanes loaded.

$\boxed{N_p = 20}$ Number of piles

$\boxed{S_{xx} = 50}$ ft^3

$\boxed{S_{yy} = 100}$ ft^3

The following illustrates the corner pile loads for 2 lanes loaded:

$$Pu2_1 := \frac{Pu2_{ftgStr1}}{N_p} + \frac{MuT2_{ftgStr1}}{S_{yy}} + \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_1 = 264.2}$$

$$Pu2_5 := \frac{Pu2_{ftgStr1}}{N_p} - \frac{MuT2_{ftgStr1}}{S_{yy}} + \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_5 = 107.46}$$

$$Pu2_{16} := \frac{Pu2_{ftgStr1}}{N_p} + \frac{MuT2_{ftgStr1}}{S_{yy}} - \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_{16} = 185.41}$$

$$Pu2_{20} := \frac{Pu2_{ftgStr1}}{N_p} - \frac{MuT2_{ftgStr1}}{S_{yy}} - \frac{MuL2_{ftgStr1\delta}}{S_{xx}} \qquad \boxed{Pu2_{20} = 28.67}$$

Pile loads between the corners can be interpolated. Similar calculations for the piles for the cases of one, two and three lanes loaded produce the following results:

$$Pu1 = \begin{pmatrix} 229.92 & 193.58 & 157.24 & 120.9 & 84.56 \\ 213.22 & 176.88 & 140.54 & 104.2 & 67.86 \\ 196.51 & 160.17 & 123.84 & 87.5 & 51.16 \\ 179.81 & 143.47 & 107.13 & 70.79 & 34.45 \end{pmatrix}$$

$$Pu2 = \begin{pmatrix} 264.2 & 225.01 & 185.83 & 146.64 & 107.46 \\ 237.93 & 198.75 & 159.57 & 120.38 & 81.2 \\ 211.67 & 172.49 & 133.3 & 94.12 & 54.94 \\ 185.41 & 146.23 & 107.04 & 67.86 & 28.67 \end{pmatrix}$$

$$Pu3 = \begin{pmatrix} 251 & 228.29 & 205.58 & 182.87 & 160.17 \\ 218.1 & 195.39 & 172.68 & 149.97 & 127.27 \\ 185.2 & 162.49 & 139.78 & 117.08 & 94.37 \\ 152.3 & 129.59 & 106.88 & 84.18 & 61.47 \end{pmatrix}$$

$\boxed{Pu1_{pile} = 229.92}$ $\boxed{Pu2_{pile} = 264.2}$ $\boxed{Pu3_{pile} = 251}$

A conservative simplification is to use the maximum pile reaction for all piles when calculating the total moment and one way shear forces on the footing.

$Pu := \max(Pu1_{pile}, Pu2_{pile}, Pu3_{pile})$ $\boxed{Pu = 264.2}$ kips

E13-1.11.1 Design for Moment

The footing is designed for moment using the pile forces computed above on a per-foot basis acting on each footing face. The design section for moment is at the face of the column. The following calculations are based on the outer row of piles in each direction, respectively.

$L_{ftg\_xx} := L_{ftg}$ $\boxed{L_{ftg\_xx} = 23}$ feet

$L_{ftg\_yy} := W_{ftg}$ $\boxed{W_{ftg} = 12}$ feet

Applied factored load per foot in the "X" direction:

$Pu_{Mom\_xx} := Pu \cdot 5$ $\boxed{Pu_{Mom\_xx} = 1320.98}$ kips

$R_{xx} := \frac{Pu_{Mom\_xx}}{L_{ftg\_xx}}$ $\boxed{R_{xx} = 57.43}$ kips per foot

Estimation of applied factored load per foot in the "Y" direction:

$Pu_{Mom\_yy} := Pu \cdot 4$ $\boxed{Pu_{Mom\_yy} = 1056.79}$ kips

$R_{yy} := \frac{Pu_{Mom\_yy}}{L_{ftg\_yy}}$ $\boxed{R_{yy} = 88.07}$ kips per foot

$arm_{xx} := 2.5$ feet

$arm_{yy} := 2.25$ feet

The moment on a per foot basis is then:

$Mu_{xx} := R_{xx} \cdot arm_{xx}$ $\boxed{Mu_{xx} = 143.59}$ kip-ft per foot

$Mu_{yy} := R_{yy} \cdot arm_{yy}$ $\boxed{Mu_{yy} = 198.15}$ kip-ft per foot

Once the maximum moment at the critical section is known, flexure reinforcement must be determined. The footing flexure reinforcement is located in the bottom of the footing and rests on top of the piles.

Assume #8 bars:

$bar\_diam8 := 1.0$ inches

$bar\_area8 := 0.79$ in^2

$\boxed{f_y = 60}$ ksi

The footing minimum tensile reinforcement requirements will be calculated. The tensile reinforcement provided must be enough to develop a factored flexural resistance at least equal to the lesser of the cracking strength or 1.33 times the factored moment from the applicable strength load combinations, **LRFD [5.7.3.3.2]**.

The cracking strength is calculated as follows, **LRFD[5.7.3.3.2]**:

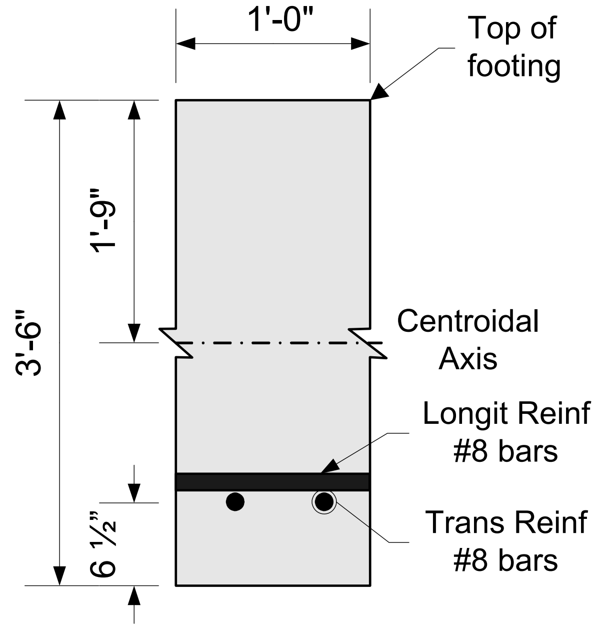

Figure E13-1.11-1
Footing Cracking Moment Dimensions

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\boxed{f_r = 0.45}$ ksi

$S_g := \dfrac{b\left(H_{ftg} \cdot 12\right)^2}{6}$ $\boxed{S_g = 3528}$ in^4

$y_t := \dfrac{H_{ftg} \cdot 12}{2}$ $\boxed{y_t = 21}$ in

$M_{cr} = \gamma_3\left(\gamma_1 \cdot f_r\right)S_g$ therefore, $M_{cr} = 1.1\left(f_r\right)S_g$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_g \cdot \dfrac{1}{12}$ $\boxed{M_{cr} = 145.21}$ kip-ft

1.33 times the factored controlling footing moment is:

$Mu_{ftg} := \max(Mu_{xx}, Mu_{yy})$ $\boxed{Mu_{ftg} = 198.15}$ kip-ft

$\boxed{1.33 \cdot Mu_{ftg} = 263.54}$ kip-ft

$M_{Design} := \min(M_{cr}, 1.33 \cdot Mu_{ftg})$ $\boxed{M_{Design} = 145.21}$ kip-ft

Mu_{ftg} exceeds M_{Design}, therefore set $M_{Design} = Mu_{ftg}$

Since the transverse moment controlled, M_{yy}, detail the transverse reinforcing to be located directly on top of the piles.

Effective depth, d_e = total footing thickness - cover - 1/2 bar diameter

$d_e := H_{ftg} \cdot 12 - Cover_{fb} - \frac{bar\_diam8}{2}$ $\boxed{d_e = 35.5}$ in

Solve for the required amount of reinforcing steel, as follows:

$\phi_f := 0.90$

$\boxed{b = 12}$ in

$\boxed{f'_c = 3.5}$ ksi

$Rn := \frac{M_{Design} \cdot 12}{\phi_f \cdot b \cdot d_e^2}$ $\boxed{Rn = 0.175}$

$\rho := 0.85 \left(\frac{f'_c}{f_y}\right) \cdot \left(1.0 - \sqrt{1.0 - \frac{2 \cdot Rn}{0.85 \cdot f'_c}}\right)$ $\boxed{\rho = 0.00300}$

$As_{ftg} := \rho \cdot b \cdot d_e$ $\boxed{As_{ftg} = 1.28}$ in^2 per foot

Required bar spacing = $\boxed{\frac{bar\_area8}{As_{ftg}} \cdot 12 = 7.41}$ in

Use #8 bars @ $bar\_space := 7$

$A_{sftg} := bar\_area8 \cdot \left(\frac{12}{bar\_space}\right)$ $\boxed{A_{sftg} = 1.35}$ in^2 per foot

Is $A_{sftg} \geq As_{ftg}$? $\boxed{check = \text{"OK"}}$

Similar calculations can be performed for the reinforcing in the longitudinal direction. The effective depth for this reinforcing is calculated based on the longitudinal bars resting directly on top of the transverse bars.

E13-1.11.2 Punching Shear Check

The factored force effects from E13-1.7 for the punching shear check at the column are:

$\boxed{Pu3_{ftgStr1} = 3124.66}$ kips

$\boxed{MuT3_{ftgStr1} = 4541.55}$ kip-ft

$\boxed{MuL3_{ftgStr1\delta} = 2467.46}$ kip-ft

$$Pu3 = \begin{pmatrix} 251 & 228.29 & 205.58 & 182.87 & 160.17 \\ 218.1 & 195.39 & 172.68 & 149.97 & 127.27 \\ 185.2 & 162.49 & 139.78 & 117.08 & 94.37 \\ 152.3 & 129.59 & 106.88 & 84.18 & 61.47 \end{pmatrix}$$

$\boxed{Pu3_{pile} = 251}$ kips

With the applied factored loads determined, the next step in the column punching shear check is to define the critical perimeter, b_o. The Specifications require that this perimeter be minimized, but need not be closer than $d_v/2$ to the perimeter of the concentrated load area. In this case, the concentrated load area is the area of the column on the footing as seen in plan.

The effective shear depth, d_v, must be defined in order to determine b_o and the punching (or two-way) shear resistance. An average effective shear depth should be used since the two-way shear area includes both the "X-X" and "Y-Y" sides of the footing. In other words, d_{ex} is not equal to d_{ey}, therefore d_{vx} will not be equal to d_{vy}. This is illustrated as follows assuming a 3'-6" footing with #8 reinforcing bars at 6" on center in both directions in the bottom of the footing:

$\boxed{b = 12}$ in

$h_{ftg} := H_{ftg} \cdot 12$ $\boxed{h_{ftg} = 42}$ in

$A_{s\_ftg} := 2 \cdot (bar\_area8)$ $\boxed{A_{s\_ftg} = 1.58}$ in^2 per foot width

Effective depth for each axis:

$\boxed{Cover_{fb} = 6}$

$d_{ey} := h_{ftg} - Cover_{fb} - \dfrac{bar\_diam8}{2}$ $\boxed{d_{ey} = 35.5}$ in

$d_{ex} := h_{ftg} - Cover_{fb} - bar\_diam8 - \dfrac{bar\_diam8}{2}$ $\boxed{d_{ex} = 34.5}$ in

Effective shear depth for each axis:

$T_{ftg} := A_{s\_ftg} \cdot f_y$ $\boxed{T_{ftg} = 94.8}$ kips

$a_{ftg} := \dfrac{T_{ftg}}{\alpha_1 \cdot f'_c \cdot b}$ $\boxed{a_{ftg} = 2.66}$ in

$d_{vx} := \max\left(d_{ex} - \dfrac{a_{ftg}}{2}, 0.9 \cdot d_{ex}, 0.72 \cdot h_{ftg}\right)$ $\boxed{d_{vx} = 33.17}$ in

$d_{vy} := \max\left(d_{ey} - \dfrac{a_{ftg}}{2}, 0.9 \cdot d_{ey}, 0.72 \cdot h_{ftg}\right)$ $\boxed{d_{vy} = 34.17}$ in

Average effective shear depth:

$d_{v\_avg} := \dfrac{d_{vx} + d_{vy}}{2}$ $\boxed{d_{v\_avg} = 33.67}$ in

With the average effective shear depth determined, the critical perimeter can be calculated as follows:

$b_{col} := L_{col} \cdot 12$ $\boxed{b_{col} = 186}$ in

$t_{col} := W_{col} \cdot 12$ $\boxed{t_{col} = 48}$ in

$b_o := 2\left[b_{col} + 2 \cdot \left(\dfrac{d_{v\_avg}}{2}\right)\right] + 2 \cdot \left[t_{col} + 2 \cdot \left(\dfrac{d_{v\_avg}}{2}\right)\right]$ $\boxed{b_o = 602.69}$ in

The factored shear resistance to punching shear is the smaller of the following two computed values:

$\beta_c := \dfrac{b_{col}}{t_{col}}$ $\boxed{\beta_c = 3.88}$

$V_{n\_punch1} := \left(0.063 + \dfrac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_{v\_avg})$ $\boxed{V_{n\_punch1} = 3626.41}$ kips

$V_{n\_punch2} := 0.126 \cdot \left(\sqrt{f'_c}\right) \cdot (b_o) \cdot (d_{v\_avg})$ $\boxed{V_{n\_punch2} = 4783.77}$ kips

$V_{n\_punch} := \min(V_{n\_punch1}, V_{n\_punch2})$ $\boxed{V_{n\_punch} = 3626.41}$ kips

$\phi_v = 0.9$

$V_{r\_punch} := \phi_v \cdot (V_{n\_punch})$ $\boxed{V_{r\_punch} = 3263.77}$ kips

With the factored shear resistance determined, the applied factored punching shear load will be computed. This value is obtained by summing the loads in the piles that are outside of the critical perimeter. As can be seen in Figure E13-1.11-2, this includes Piles 1 through 5, 6, 10,11, 15, and 16 through 20. These piles are entirely outside of the critical perimeter. If part of a pile is inside the critical perimeter, then only the portion of the pile load outside the critical perimeter is used for the punching shear check, **LRFD [5.13.3.6.1]**.

$$\left(\frac{t_{col}}{2} + \frac{d_{v\_avg}}{2}\right) \cdot \frac{1}{12} = 3.4 \quad \text{feet}$$

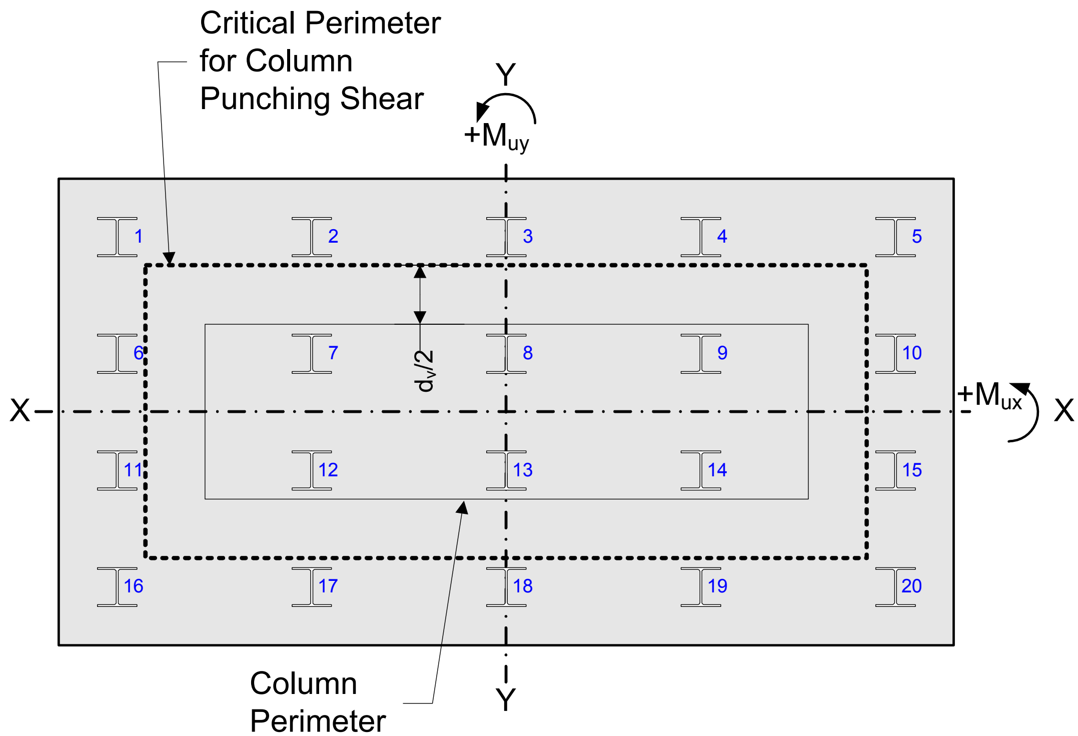

Figure E13-1.11-2
Critical Perimeter for Column Punching Shear

The total applied factored shear used for the punching shear check is the sum of the piles outside of the shear perimeter (1 through 5, 6, 10, 11, 15 and 16 through 20):

$$V_{u\_punch} := \max\left(Pu1_{punch\_col}, Pu2_{punch\_col}, Pu3_{punch\_col}\right)$$

$\boxed{V_{u\_punch} = 2187.26}$ kips

$\boxed{V_{r\_punch} = 3263.77}$ kips

$V_{u\_punch} \leq V_{r\_punch}$

$\boxed{check = \text{"OK"}}$

For two-way action around the maximum loaded pile, the pile critical perimeter, b_o, is located a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge.

Two-way action should be checked for the maximum loaded pile, The effective shear depth, d_v is the same as that used for the punching shear check for the column.

$V_{u2way} := Pu2_{pile}$ $\boxed{V_{u2way} = 264.2}$ kips

$\boxed{d_{v\_avg} = 33.67}$ in

$\boxed{0.5 \cdot d_{v\_avg} = 16.84}$ in

Two-way action or punching shear resistance for sections without transverse reinforcement can then be calculated as follows:

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot b_o \cdot d_v \le 0.126 \cdot \sqrt{f'_c} \cdot b_o \cdot d_v$$

$\boxed{B_{xx} = 12.05}$ in

$\boxed{B_{yy} = 11.78}$ in

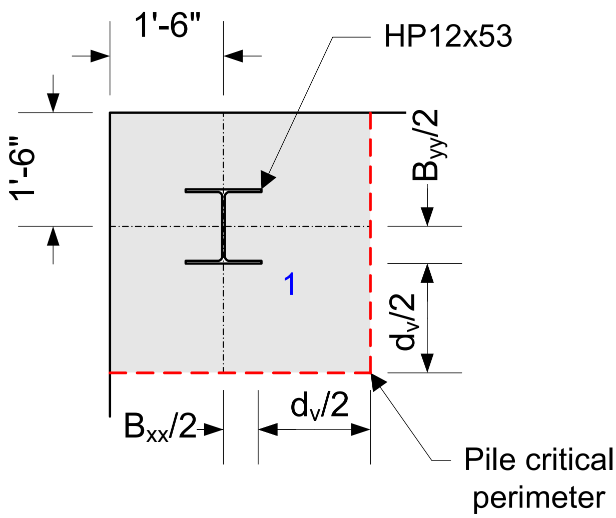

Figure E13-1.11-3
Pile Two-way Action Critical Perimeter

Since the critical section is outside of the footing, only include the portion of the shear perimeter that is located within the footing:

$b_{o\_xx} := 1.5 \cdot 12 + \frac{B_{xx}}{2} + \frac{d_{v\_avg}}{2}$ $\boxed{b_{o\_xx} = 40.86}$ in

$b_{o\_yy} := 1.5 \cdot 12 + \frac{B_{yy}}{2} + \frac{d_{v\_avg}}{2}$ $\boxed{b_{o\_yy} = 40.73}$ in

Ratio of long to short side of critical perimeter:

$\beta_{c\_pile} := \dfrac{b_{o\_xx}}{b_{o\_yy}}$ $\boxed{\beta_{c\_pile} = 1.003}$

$b_{o\_pile} := b_{o\_xx} + b_{o\_yy}$ $\boxed{b_{o\_pile} = 81.59}$ in

$V_{n\_pile1} := \left(0.063 + \dfrac{0.126}{\beta_{c\_pile}}\right)\cdot\sqrt{f'_c}\cdot(b_{o\_pile})\cdot(d_{v\_avg})$ $\boxed{V_{n\_pile1} = 969.24}$ kips

$V_{n\_pile2} := 0.126\cdot\left(\sqrt{f'_c}\right)\cdot(b_{o\_pile})\cdot(d_{v\_avg})$ $\boxed{V_{n\_pile2} = 647.59}$ kips

$V_{n\_pile} := \min(V_{n\_pile1}, V_{n\_pile2})$ $\boxed{V_{n\_pile} = 647.59}$ kips

$\phi_v = 0.9$

$V_{r\_pile} := \phi_v\cdot(V_{n\_pile})$ $\boxed{V_{r\_pile} = 582.83}$ kips

$\boxed{V_{u2way} = 264.2}$ kips

$V_{r\_pile} \geq V_{u2way}$

$\boxed{check = \text{"OK"}}$

E13-1.11.3 One Way Shear Check

Design for one way shear in both the transverse and longitudinal directions.

For one way action in the pier footing, in accordance with **LRFD[5.13.3.6.1]** & **[5.8.3.2]** the critical section is taken as the larger of:

$0.5\cdot d_v\cdot \cot\theta$ or d_v

$\theta := 45deg$

The term d_v is calculated the same as it is for the punching shear above:

$\boxed{d_{vx} = 33.17}$ in

$\boxed{d_{vy} = 34.17}$ in

Now the critical section can be calculated:

$dv_{xx} := \max(0.5\cdot d_{vx}\cdot\cot(\theta), d_{vx})$ $\boxed{dv_{xx} = 33.17}$ in

$dv_{yy} := \max(0.5\cdot d_{vy}\cdot\cot(\theta), d_{vy})$ $\boxed{dv_{yy} = 34.17}$ in

Distance from face of column to CL of pile in longitudinal and transverse directions:

$\boxed{arm_{xx} = 2.5}$ feet

$\boxed{arm_{yy} = 2.25}$ feet

Distance from face of column to outside edge of pile in longitudinal and transverse directions:

$\boxed{arm_{xx} \cdot 12 + \frac{B_{yy}}{2} = 35.89}$ > d_{vx}, design check required

$\boxed{arm_{yy} \cdot 12 + \frac{B_{xx}}{2} = 33.02}$ < d_{vy}, no design check required

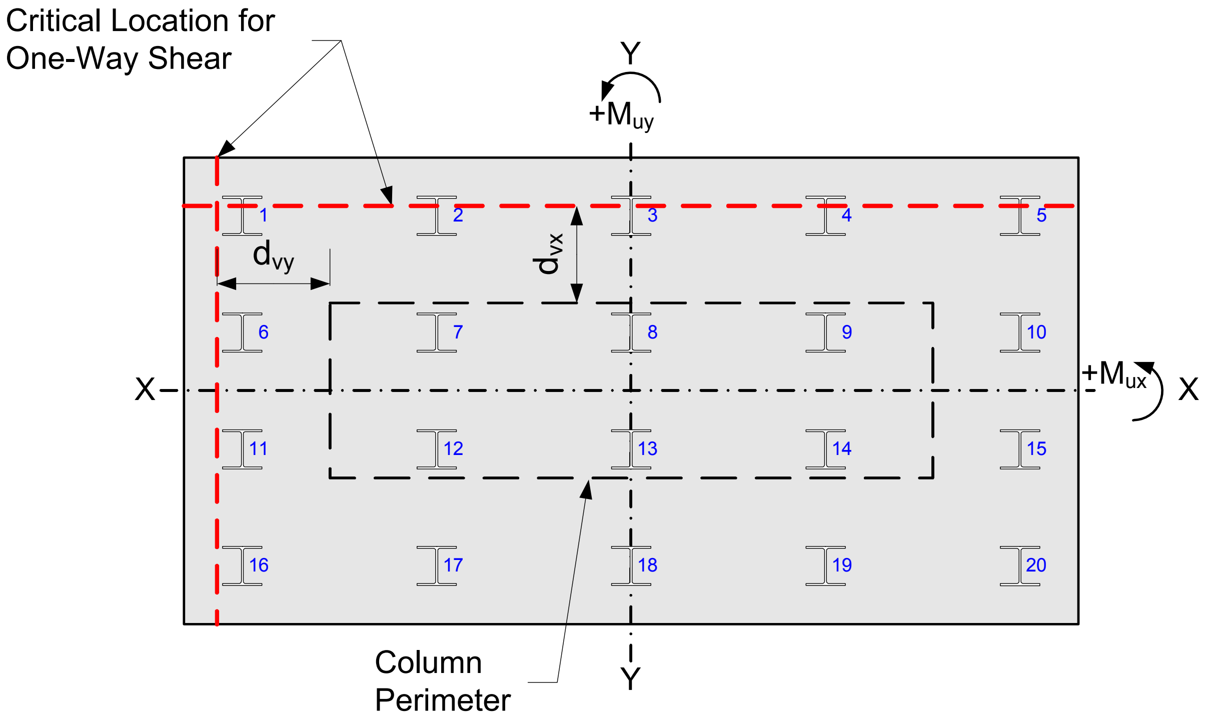

Figure E13-1.11-4
Critical Section for One-Way Shear

Portion of pile outside of the critical section for one way shear in the longitudinal direction:

$$b_{xx} := arm_{xx} \cdot 12 + \frac{B_{yy}}{2} - d_{vx}$$ $\boxed{b_{xx} = 2.72}$ inches

The load applied to the critical section will be based on the proportion of the pile located outside of the critical section. As a conservative estimate, the maximum pile reaction will be assumed for all piles.

$\boxed{Pu = 264.2}$ kips

$Pu_{1wayx} := Pu \cdot 5$ $\boxed{Pu_{1wayx} = 1320.98}$ kips

$Vu_{1wayx} := Pu_{1wayx} \cdot \frac{b_{xx}}{B_{yy}}$ $\boxed{Vu_{1wayx} = 304.76}$ kips

The nominal shear resistance shall be calculated in accordance with **LRFD [5.8.3.3]** and is the lesser of the following:

$\beta_{1way} := 2.0$

$b_v := L_{ftg} \cdot 12$ $\boxed{b_v = 276}$ inches

$V_{n\_1way1} := 0.0316 \cdot \beta_{1way} \cdot \sqrt{f'_c} \cdot (b_v) \cdot (d_{vx})$ $\boxed{V_{n\_1way1} = 1082.52}$ kips

$V_{n\_1ay2} := 0.25 \cdot (f'_c) \cdot (b_v) \cdot (d_{vx})$ $\boxed{V_{n\_1ay2} = 8011.1}$ kips

$V_{n\_1way} := \min(V_{n\_1way1}, V_{n\_1ay2})$ $\boxed{V_{n\_1way} = 1082.52}$ kips

$\phi_v = 0.9$

$V_{r\_1way} := \phi_v \cdot (V_{n\_1way})$ $\boxed{V_{r\_1way} = 974.27}$ kips

$\boxed{Vu_{1wayx} = 304.76}$ kips

$V_{r\_1way} \geq Vu_{1wayx}$

$\boxed{check = "OK"}$

E13-1.12 Final Pier Schematic

Figure E13-1.12-1 shows the final pier dimensions along with the required reinforcement in the pier cap and column.

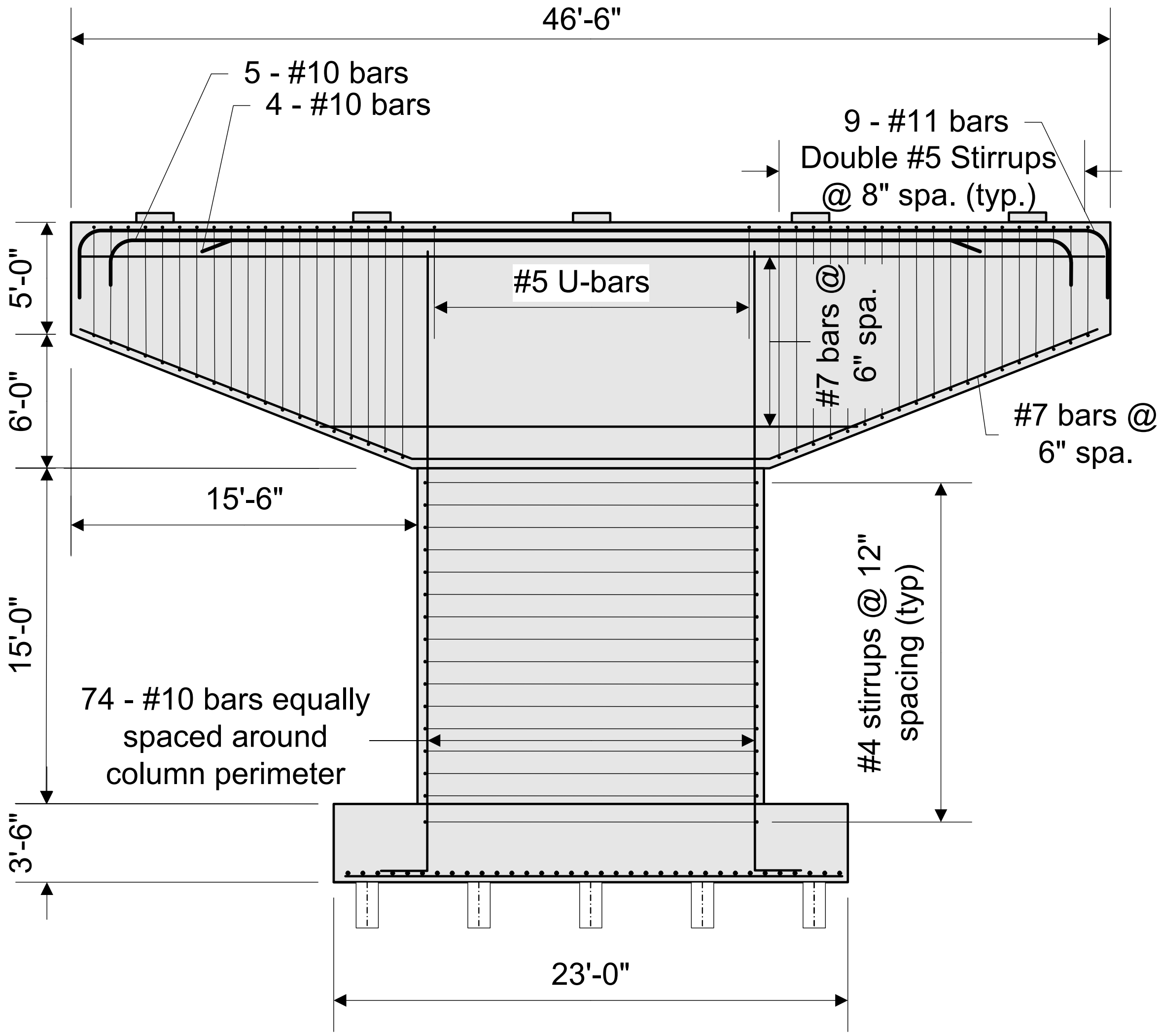

Figure E13-1.12-1
Final Pier Design

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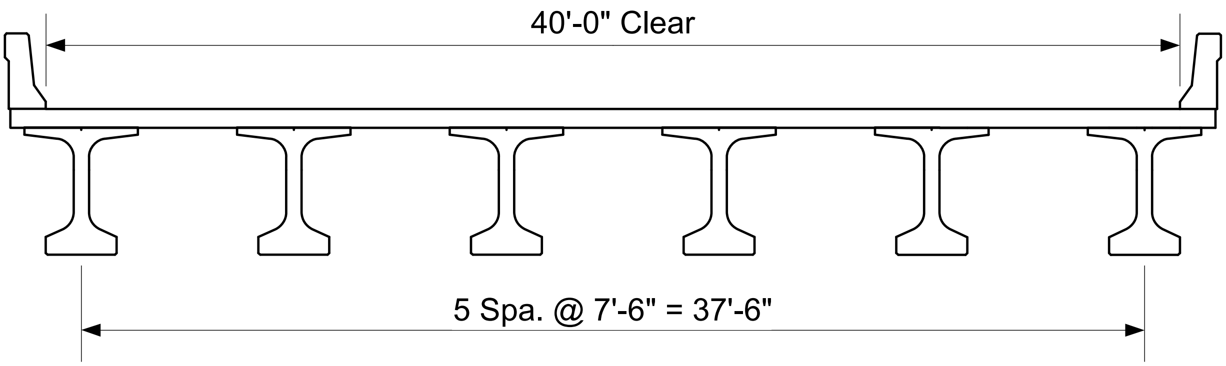
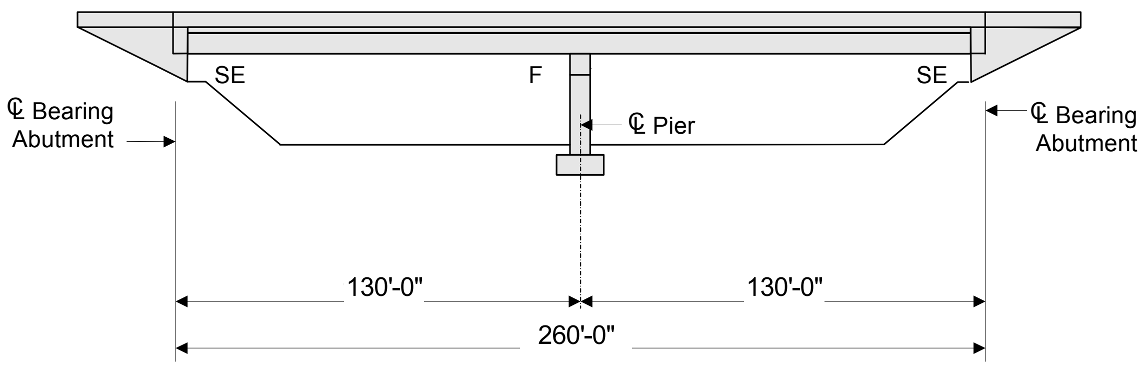
Table of Contents

E13-2 Multi-Column Pier Design Example - LRFD

2 Span Bridge, 54W, LRFD Design



This pier is designed for the superstructure as detailed in example **E19-2**. This is a two-span prestressed girder grade separation structure. Semi-expansion bearings are located at the abutments, and fixed bearings are used at the pier.



E13-2.1 Obtain Design Criteria

This multi-column pier design example is based on ***AASHTO LRFD Bridge Design Specifications, (Seventh Edition - 2016 Interim).*** The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice. Calculations are only shown for the pier cap. For example column and footing calculations, see example E13-1.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-2.1.1 Material Properties:

$w_c := 0.150$ Concrete density, kcf

$f'_c := 3.5$ Concrete 28-day compressive strength, ksi **LRFD [5.4.2.1 & Table C5.4.2.1-1]**

$f_y := 60$ Reinforcement strength, ksi **LRFD [5.4.3 & 6.10.1.7]**

$E_s := 29000$ Modulus of Elasticity of the reinforcing steel, ksi

$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ **LRFD [C5.4.2.4]**

$\boxed{E_c = 3587}$ Modulus of Elasticity of the Concrete, ksi

E13-2.1.2 Reinforcing steel cover requirements (assume epoxy coated bars)

Cover dimension listed below is in accordance with **LRFD [Table 5.12.3-1].**

$Cover_{cap} := 2.5$ Concrete cover in pier cap, inches

E13-2.1.3 Relevant Superstructure Data

$L := 130$ design span length, feet

$w_b := 42.5$ out to out width of deck, feet

$w_{deck} := 40$ clear width of deck, feet

$w_p := 0.387$ weight of Wisconsin Type LF parapet, klf

$t_s := 8$ slab thickness, inches

$t_{haunch} := 4$ haunch thickness, inches

$skew := 0$ skew angle, degrees

$S := 7.5$ girder spacing, ft

$ng := 6$ number of girders

$DOH := \dfrac{w_b - (ng - 1) \cdot S}{2}$ deck overhang length $\boxed{DOH = 2.5}$ feet

$w_{tf} := 48$ width of 54W girder top flange, inches

$t_{tf} := 3$ thickness of 54W girder top flange, inches

$tf_{slope} := \frac{2.5}{20.75}$ slope of bottom surface of top flange $\boxed{tf_{slope} = 0.12}$ feet per foot

$girder_H := 54$ height of 54W girder, inches

E13-2.1.4 Select Optimum Pier Type

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. The most common pier types are single column (i.e., "hammerhead"), solid wall type, and bent type (multi-column or pile bent). For this design example, a multi-column pier was chosen.

E13-2.1.5 Select Preliminary Pier Dimensions

Since the Specifications do not have standards regarding maximum or minimum dimensions for a pier cap, column, or footing, the designer should base the preliminary pier dimensions on state specific standards, previous designs, and past experience. The pier cap, however, must be wide enough to accommodate the bearing.

$cap_L := 41.5$ overall cap length, ft

$cap_H := 4.0$ pier cap height, ft

$cap_W := 3.5$ pier cap width, ft

$col_{spa} := 18.25$ column spacing, ft

$col_d := 3$ column depth (perpendicular to pier CL), ft

$col_w := 4$ column width (parallel to pier CL), ft

$col_h := 18$ column height, ft

$cap_{OH} := 2.5$ pier cap overhang dimension, ft

Figures E13-2.1-1 and E13-2.1-2 show the preliminary dimensions selected for this pier design example.

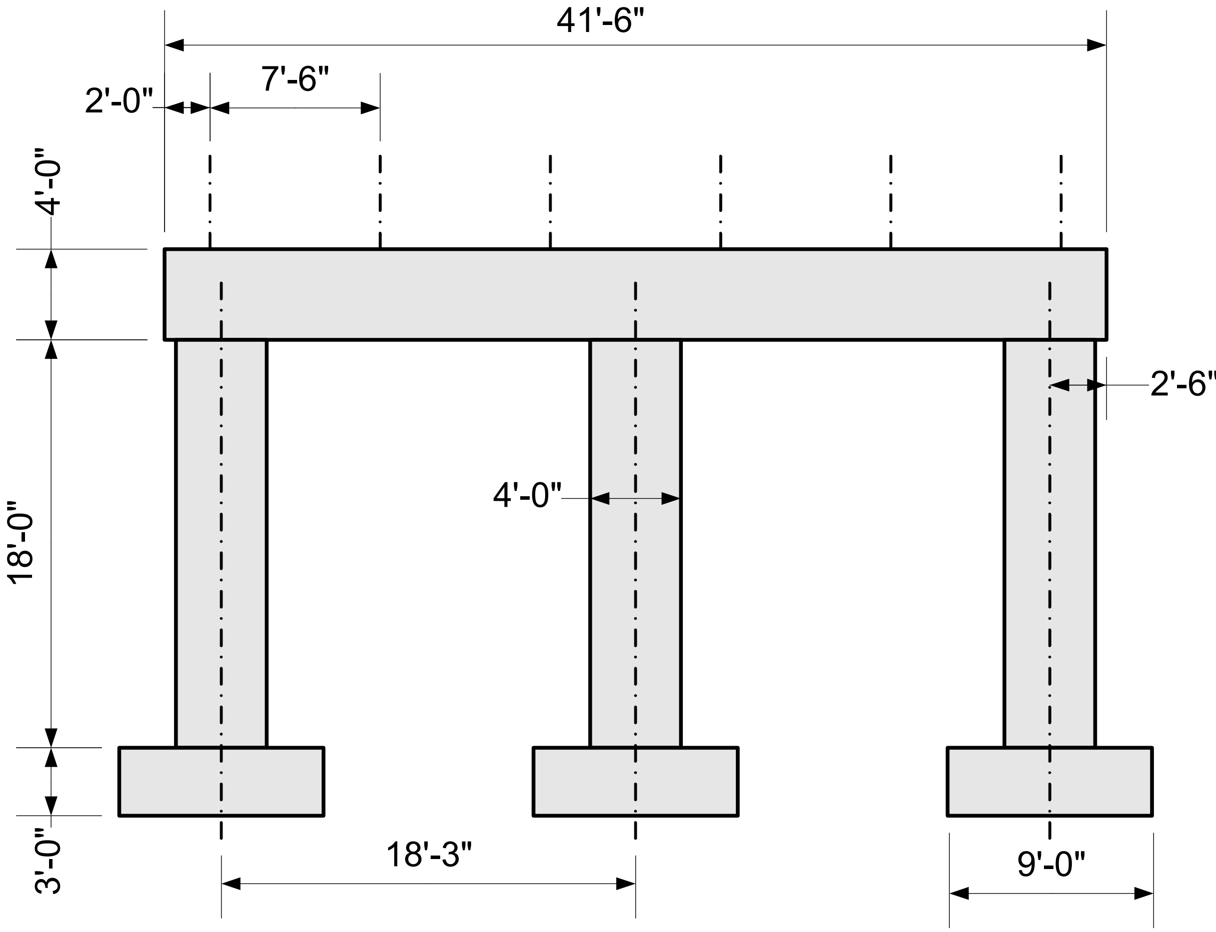


Figure E13-2.1-1
Preliminary Pier Dimensions - Front Elevation

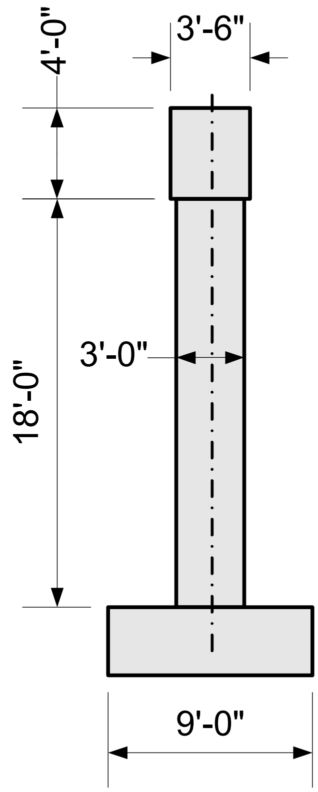


Figure E13-2.1-2
Preliminary Pier Dimensions - End Elevation

E13-2.2 Loads

$w_g := 0.831$ weight of 54W girder, klf

$w_{deck\_int} := w_c \cdot \frac{t_s \cdot S}{12}$ weight of deck slab (int), klf $\boxed{w_{deck\_int} = 0.75}$ klf

$OH := DOH - \frac{w_{tf}}{2 \cdot 12}$ deck overhang projection, ft $\boxed{OH = 0.5}$ ft

weight of deck slab (ext), klf

$$w_{deck\_ext} := w_c \cdot \left[\frac{t_s}{12} \cdot \left(\frac{S}{2} + DOH\right) + \frac{1}{2} \cdot (OH) \cdot \left(\frac{t_{haunch} + t_{tf}}{12^2} - OH \cdot tf_{slope} \cdot \frac{1}{2}\right)\right]$$

$\boxed{w_{deck\_ext} = 0.63}$ klf

weight of haunch, klf

$w_h := w_c \cdot \frac{t_{haunch} \cdot w_{tf}}{12^2}$ $\boxed{w_h = 0.2}$ klf

$w_{diaph\_int} := 0.410$ weight of diaphragms on interior girder (assume 2), kips

$w_{diaph\_ext} := 0.205$ weight of diaphragms on exterior girder, kips

$w_{ws} := 0.020$ future wearing surface, ksf

$w_p := 0.387$ weight of each parapet, klf

weight of concrete diaphragm between exterior girders

$w_{diaph} := w_c \cdot \frac{girder_H}{12} \cdot 2$ $\boxed{w_{diaph} = 1.35}$ klf

weight of cap

$w_{cap} := w_c \cdot cap_W \cdot cap_H$ $\boxed{w_{cap} = 2.1}$ klf

E13-2.2.1 Superstructure Dead Loads

DC Loads and Reactions

Interior DC1,DC2 and DW Loads

$w_{DC1\_int} := w_g + w_{deck\_int} + w_h + w_{diaph\_int}$ $\boxed{w_{DC1\_int} = 2.19}$ klf

$$w_{DC2} := \frac{2 \cdot w_p}{ng}$$ $\boxed{w_{DC2} = 0.13}$ klf

$$w_{DW} := \frac{w_{ws} \cdot w_{deck}}{ng}$$ $\boxed{w_{DW} = 0.13}$ klf

Interior DC and DW Reactions

$$R_{DCi} := \left(\frac{1}{2} \cdot L \cdot w_{DC1\_int} + \frac{5}{8} \cdot L \cdot w_{DC2}\right) \cdot 2$$ $\boxed{R_{DCi} = 305.79}$ kips

$$R_{DWi} := \left(\frac{5}{8} \cdot L \cdot w_{DW}\right) \cdot 2$$ $\boxed{R_{DWi} = 21.67}$ kips

Exterior DC1 Loads

$$w_{DC1\_ext} := w_g + w_{deck\_ext} + w_h + w_{diaph\_ext}$$ $\boxed{w_{DC1\_ext} = 1.86}$ klf

Note: DC2 and DW loads are the same for interior and exterior girders.

Exterior DC and DW Reactions

$$R_{DCe} := \left(\frac{1}{2} \cdot L \cdot w_{DC1\_ext} + \frac{5}{8} \cdot L \cdot w_{DC2}\right) \cdot 2$$ $\boxed{R_{DCe} = 262.98}$ kips

$$R_{DWe} := \left(\frac{5}{8} \cdot L \cdot w_{DW}\right) \cdot 2$$ $\boxed{R_{DWe} = 21.67}$ kips

The unfactored dead load reactions are listed below:

Unfactored Girder Reactions (kips)		
Girder #	DC	DW
1	263.0	21.7
2	305.8	21.7
3	305.8	21.7
4	305.8	21.7
5	305.8	21.7
6	263.0	21.7

Table E13-2.2-1
Unfactored Girder Dead Load Reactions

E13-2.2.2 Live Load Reactions per Design Lane

From girder line analysis, the following pier unfactored live load reactions are obtained:

$TruckPair := 125.64$ kips per design lane

$Lane := 103.94$ kips per design lane

$DLA := 1.33$ dynamic load allowance

These loads are per design lane and do not include dynamic load allowance. The pier reactions are controlled by the 90% (Truck Pair + Lane) loading condition. The resulting combined live load reactions per design lane (including dynamic load allowance) are:

$$R\_LL_{DesLane} := 0.90 \cdot (TruckPair \cdot DLA + Lane) \qquad \boxed{R\_LL_{DesLane} = 243.94} \text{ kips}$$

The resulting wheel loads are:

$$R_{LLw} := \frac{0.90 \cdot TruckPair \cdot DLA}{2} \qquad \boxed{R_{LLw} = 75.2} \text{ kips per wheel}$$

$$R_{LLlane} := \frac{0.90 \cdot Lane}{10} \qquad \boxed{R_{LLlane} = 9.35} \text{ kips per foot}$$

E13-2.2.3 Superstructure Live Load Reactions

For the pier in this design example, the maximum live load effects in the pier cap, column and footing are based on either one, two or three lanes loaded (whichever results in the worst force effect). The lanes are moved across the deck to create the envelope of force effects. The following figures illustrate the lane locations loaded to determine the maximum positive and negative moments as well as the maximum shear force effects in the pier cap.

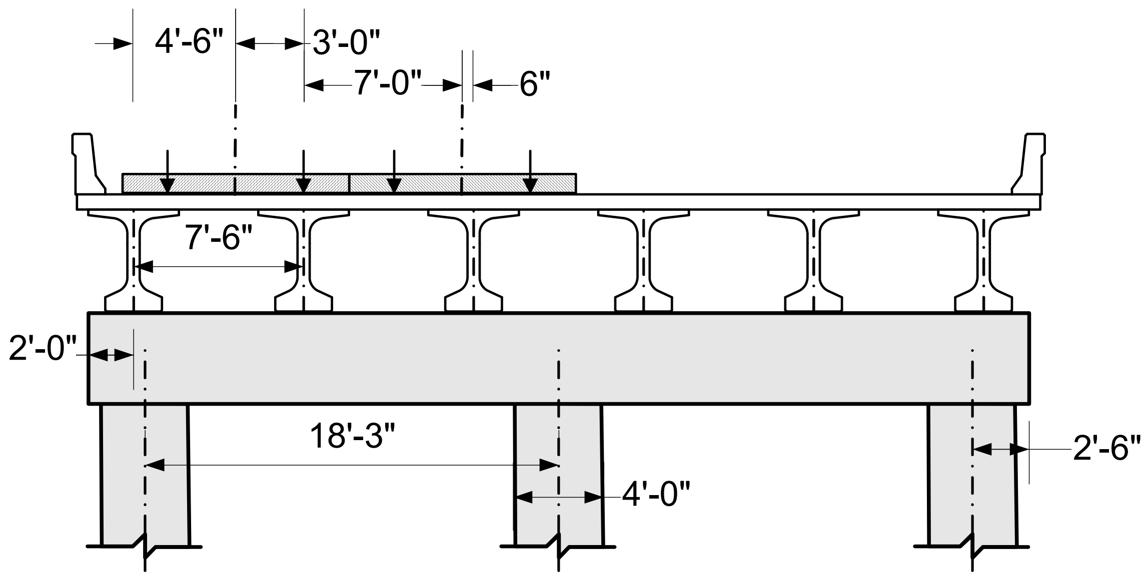


Figure E13-2.2-1
Lane Locations for Maximum Positive Moment

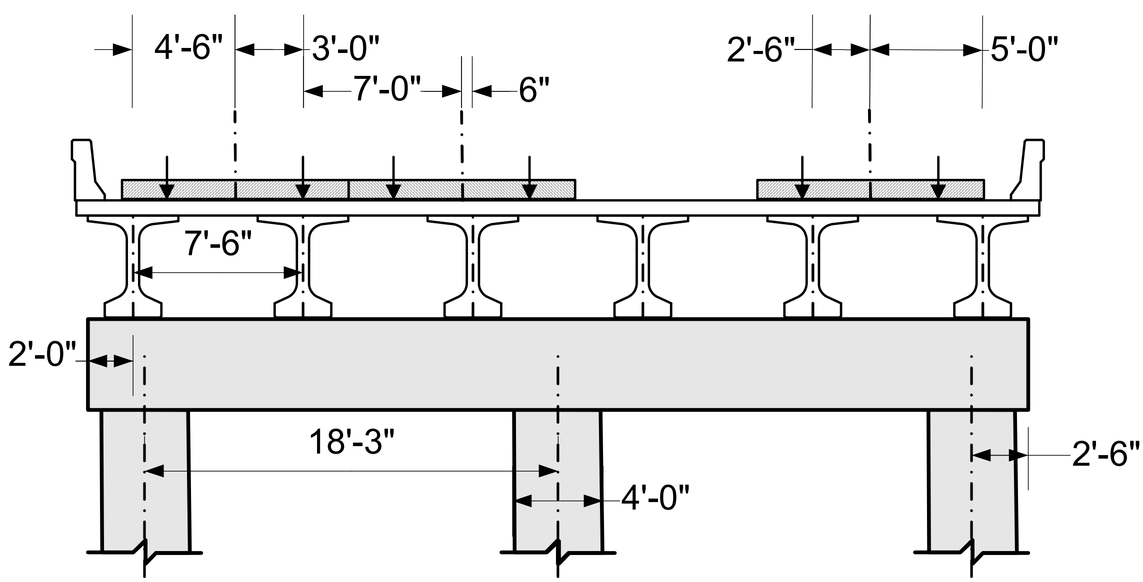


Figure E13-2.2-2
Lane Locations for Maximum Negative Moment

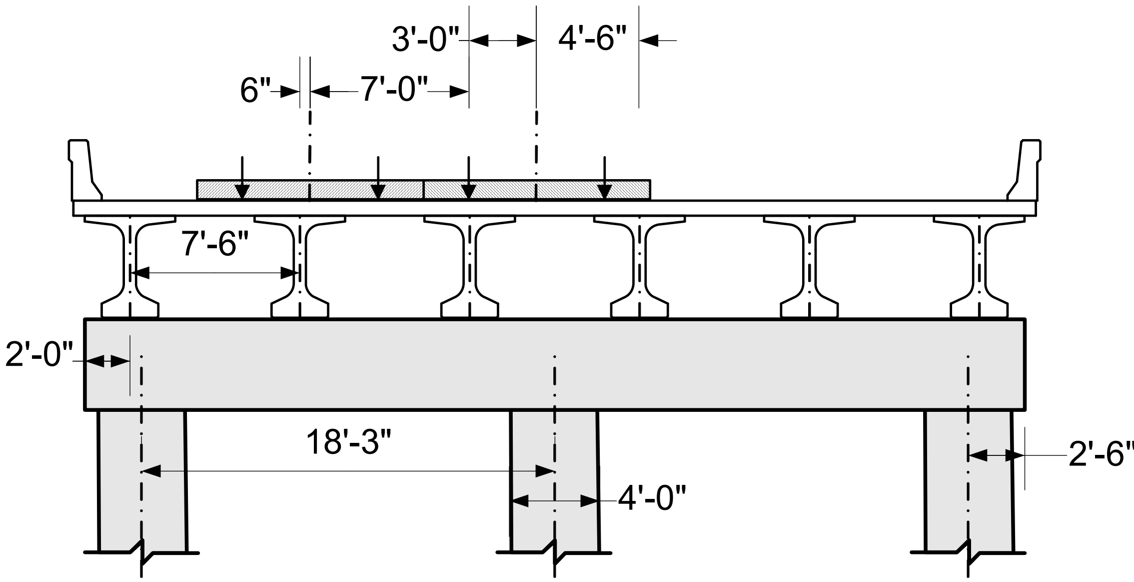


Figure E13-2.2-3
Lane Locations for Maximum Shear

The next step is to compute the reactions due to the above loads at each of the six bearing locations. This is generally carried out by assuming the deck is pinned (i.e., discontinuous) at the interior girder locations but continuous over the exterior girders. Solving for the reactions is then elementary. The computations for the reactions for maximum moment with only 2 lanes loaded are illustrated below as an example. All reactions shown are in kips.

$m_2 := 1.0$ Multi-presence factor for two lanes loaded

$$R1_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{6.0}{7.5} \right) + R_{LLlane} \cdot \left(0.5 + \frac{7.5}{2} \right) \right] \qquad \boxed{R1_{LL} = 99.91}$$

$$R2_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{1.5}{7.5} + 1 + \frac{3.5}{7.5} \right) + R_{LLlane} \cdot (7.5) \right] \qquad \boxed{R2_{LL} = 195.49}$$

$$R3_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{4.0 + 5.0}{7.5} \right) + R_{LLlane} \cdot \left(\frac{7.5}{2} + 4.5 \cdot \frac{5.25}{7.5} \right) \right] \qquad \boxed{R3_{LL} = 154.78}$$

$$R4_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{2.5}{7.5} \right) + R_{LLlane} \cdot 4.5 \cdot \frac{2.25}{7.5} \right] \qquad \boxed{R4_{LL} = 37.69}$$

$$R5_{LL} := 0 \qquad \boxed{R5_{LL} = 0}$$

$$R6_{LL} := 0 \qquad \boxed{R6_{LL} = 0}$$

E13-2.3 Unfactored Force Effects

The resulting unfactored force effects for the load cases shown above are shown in the table below. Note that the maximum shear and negative moment values are taken at the face of the column.

Unfactored Force Effects			
Effect	DC	DW	LL
Maximum Positive Moment	943.1	62.17	628.4
Maximum Negative Moment	-585.6	-39.03	-218.9
Maximum Shear	429.2	28.53	228.3
(Corresponding Moment)	-585.6	-39.03	-119.3

Table E13-2.3-1
Unfactored Force Effects

E13-2.4 Load Factors

From **LRFD [Table 3.4.1-1]**:

DC DW LL

$\gamma st_{DC} := 1.25$ $\gamma st_{DW} := 1.50$ $\gamma st_{LL} := 1.75$

$\gamma s1_{DC} := 1.0$ $\gamma s1_{DW} := 1.0$ $\gamma s1_{LL} := 1.0$

E13-2.5 Combined Force Effects

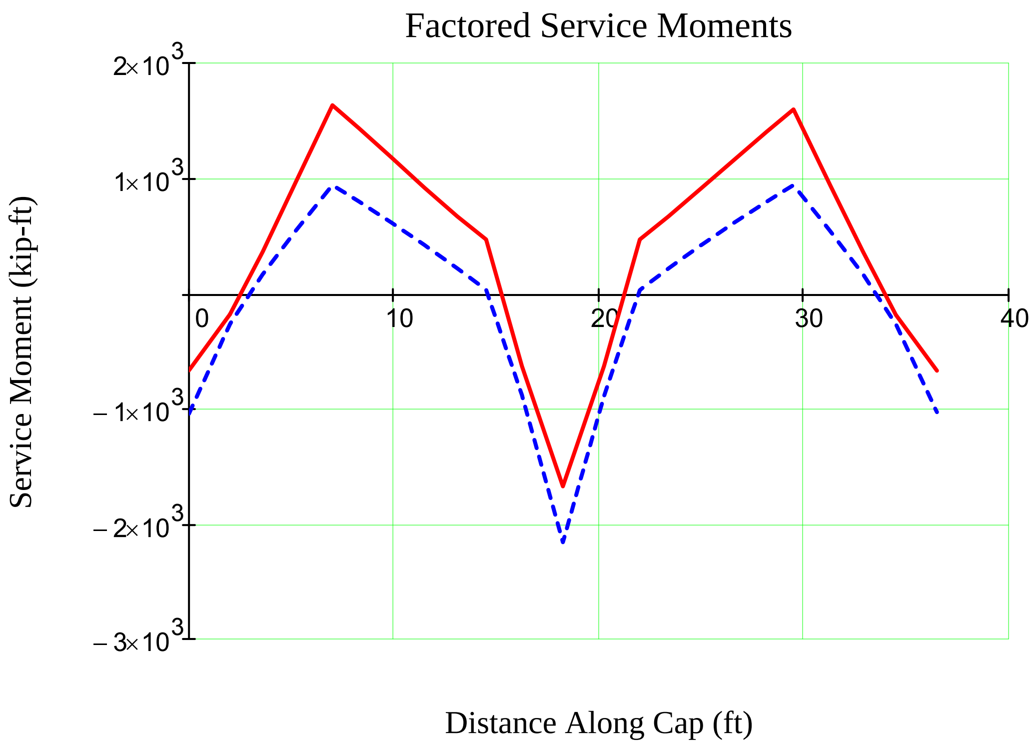
The resulting factored Service and Strength force effects for the load cases previously illustrated are shown in the tables below. The full Service and Strength factored moment and shear envelopes are shown in the following graphs.

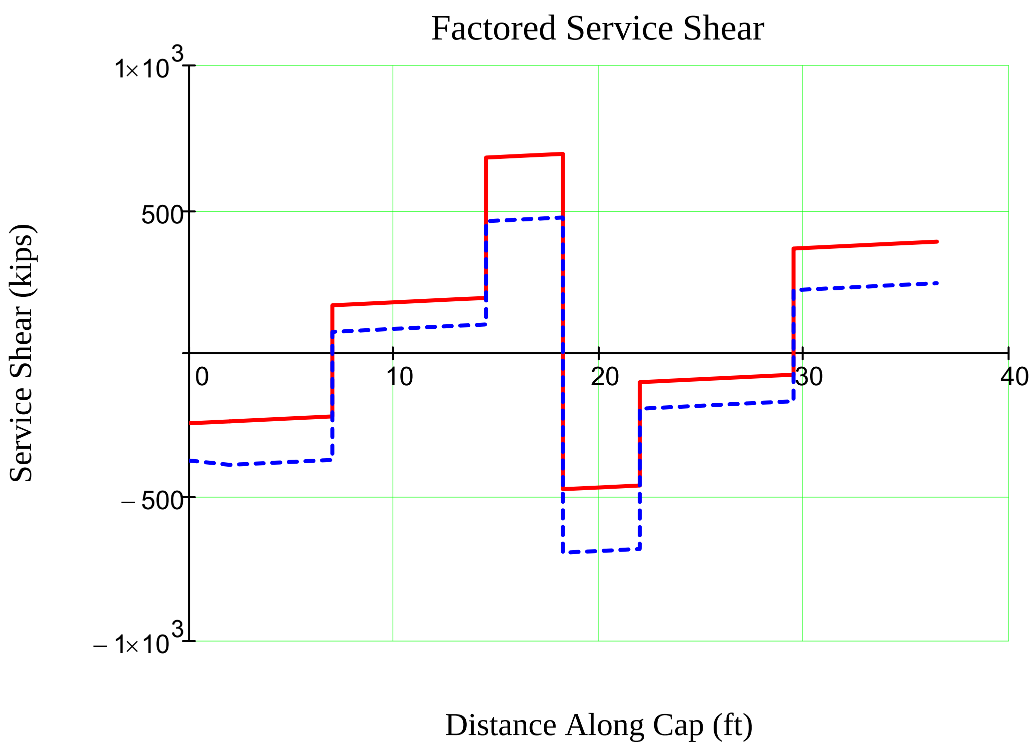
Factored Service Force Effects				
Effect	DC	DW	LL	Total
Maximum Positive Moment	943.1	62.2	628.4	1633.7
Maximum Negative Moment	-585.6	-39.0	-218.9	-843.5
Maximum Shear	429.2	28.5	228.3	686.0
(Corresponding Moment)	-585.6	-39.0	-119.3	-743.9

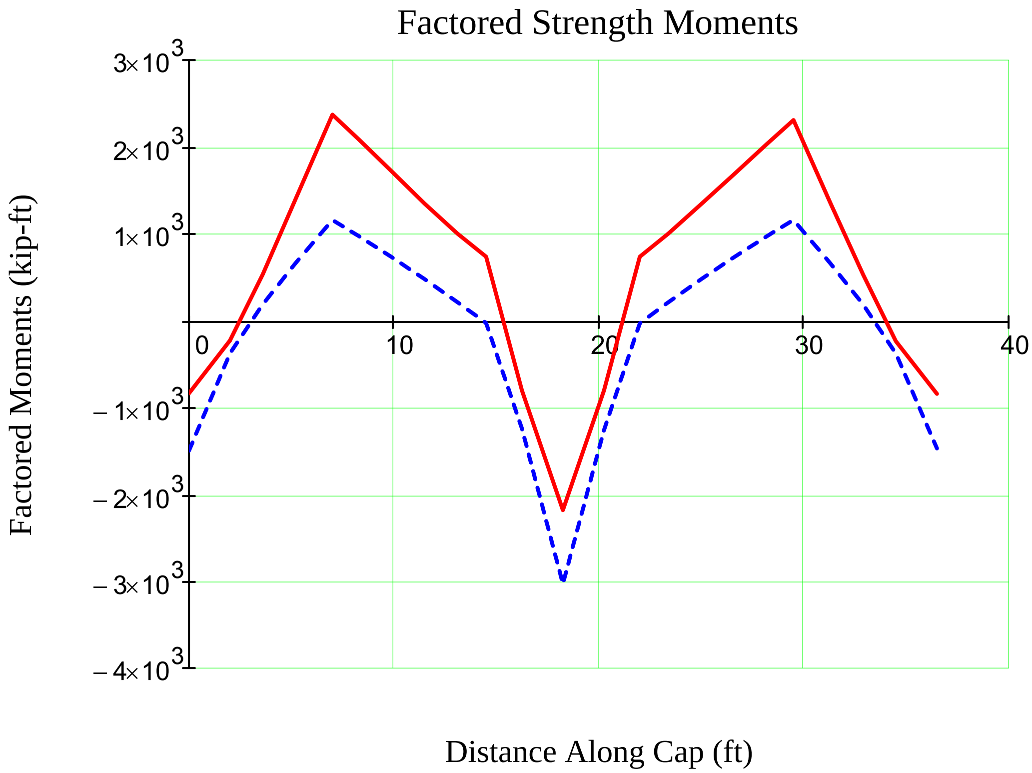
Table E13-2.5-1
Factored Service Force Effects

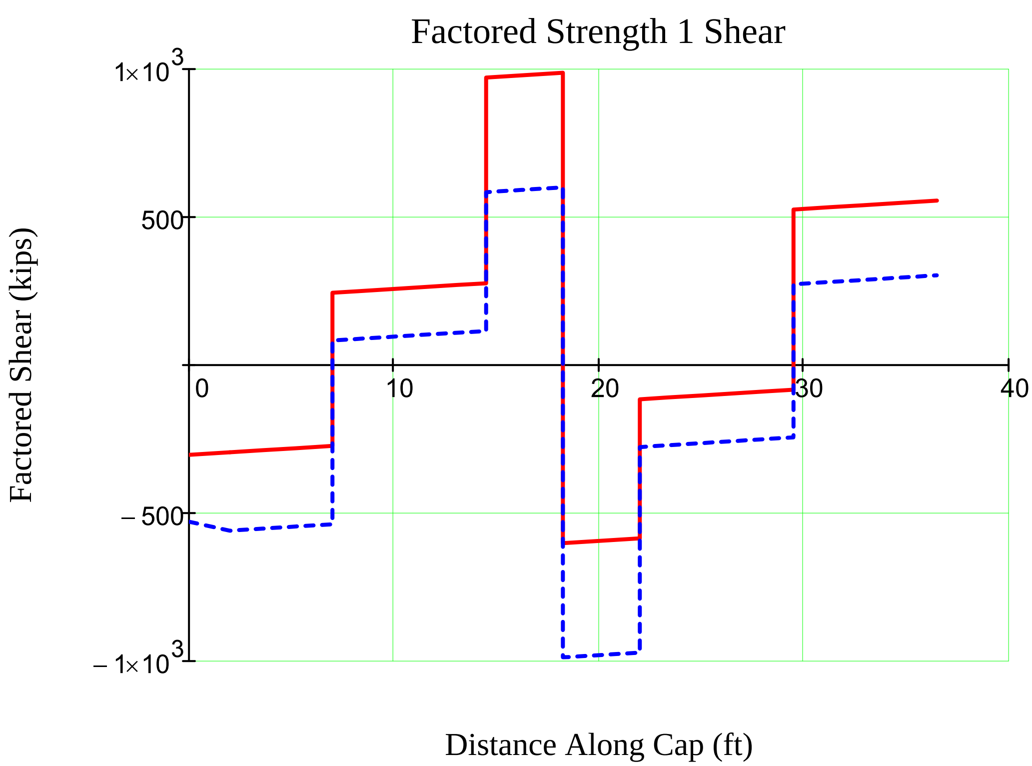
Factored Strength Force Effects				
Effect	DC	DW	LL	Total
Maximum Positive Moment	1178.9	93.3	1099.7	2371.8
Maximum Negative Moment	-732.0	-58.5	-383.1	-1173.6
Maximum Shear	536.5	42.8	399.5	978.8
(Corresponding Moment)	-732.0	-58.5	-208.8	-999.3

Table E13-2.5-2
Factored Strength I Force Effects


Factored Service Moments
2×10^3
1×10^3
−1×10^3
−2×10^3
−3×10^3
0
10
20
30
40
Service Moment (kip-ft)
Distance Along Cap (ft)


Factored Service Shear
1×10^3
500
−500
−1×10^3
0
10
20
30
40
Service Shear (kips)
Distance Along Cap (ft)


Factored Strength Moments
3×10^3
2×10^3
1×10^3
0
10
20
30
40
− 1×10^3
− 2×10^3
− 3×10^3
− 4×10^3
Factored Moments (kip-ft)
Distance Along Cap (ft)


Factored Strength 1 Shear
1×10^3
500
0
10
20
30
40
− 500
− 1×10^3
Factored Shear (kips)
Distance Along Cap (ft)

E13-2.6 Pier Cap Design

Calculate positive and negative moment requirements.

E13-2.6.1 Positive Moment Capacity Between Columns

It is assumed that there will be two layers of positive moment reinforcement. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

In accordance with **LRFD [5.10.3.1.3]** the minimum clear space between the bars in layers is one inch or the nominal diameter of the bars.

$spa_{clear} := 1.75$ in

$bar_{stirrup} := 5$ (transverse bar size)

$\boxed{Bar_D(bar_{stirrup}) = 0.63}$ in (transverse bar diameter)

$Bar_{No\_pos} := 9$

$\boxed{Bar_D(Bar_{No\_pos}) = 1.13}$ in (Assumed bar size)

$$d_e := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - Bar_D(Bar_{No\_pos}) - \frac{spa_{clear}}{2}$$

$\boxed{d_e = 42.87}$ in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the cap:

$b_w := cap_W \cdot 12$ $\boxed{b_w = 42}$ in

$\boxed{Mu_{pos} = 2372}$ kip-ft

$$R_u := \frac{Mu_{pos} \cdot 12}{\phi_f \cdot b_w \cdot d_e^2}$$ $\boxed{R_u = 0.4097}$ ksi

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.00738}$

$A_s := \rho \cdot b_w \cdot d_e$ $\boxed{A_s = 13.28}$ in^2

This requires $n_{bars\_pos} := 14$ bars. Use $n_{bars\_pos1} := 9$ bars in the bottom layer and $n_{bars\_pos2} := 5$ bars in the top layer. Check spacing requirements.

$$spa_{pos} := \frac{b_w - 2 \cdot (cover + Bar_D(bar_{stirrup})) - Bar_D(Bar_{No\_pos})}{n_{bars\_pos1} - 1}$$

$\boxed{spa_{pos} = 4.33}$ in

$clear_{spa} := spa_{pos} - Bar_D(Bar_{No\_pos})$ $\boxed{clear_{spa} = 3.2}$ in

The minimum clear spacing is equal to 1.5 times the maximum aggregate size of 1.5 inches.

$spa_{min} := 1.5 \cdot 1.5$ $\boxed{spa_{min} = 2.25}$ in

Is $spa_{min} \le clear_{spa}$? $\boxed{check = \text{"OK"}}$

$As_{prov\_pos} := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos}$ $\boxed{As_{prov\_pos} = 14}$ in^2

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi)

$a := \dfrac{As_{prov\_pos} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$ $\boxed{a = 6.72}$ in

$Mn_{pos} := As_{prov\_pos} \cdot f_y \cdot \left(d_e - \dfrac{a}{2}\right) \cdot \dfrac{1}{12}$ $\boxed{Mn_{pos} = 2766}$ kip-ft

$Mr_{pos} := \phi_f \cdot Mn_{pos}$ $\boxed{Mr_{pos} = 2489}$ kip-ft

$\boxed{Mu_{pos} = 2372}$ kip-ft

Is $Mu_{pos} \le Mr_{pos}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$S_{cap} := \dfrac{(cap_W \cdot 12) \cdot (cap_H \cdot 12)^2}{6}$ $\boxed{S_{cap} = 16128}$ in^3

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.45}$ ksi

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_{cap}$ therefore, $M_{cr} = 1.1(f_r)S_{cap}$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 \cdot f_r \cdot S_{cap} \cdot \dfrac{1}{12}$ $\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{pos} = 3155}$ kip-ft

Is Mr_{pos} greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{pos}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho := \dfrac{As_{prov\_pos}}{b_w \cdot d_e}$ $\boxed{\rho = 0.00778}$

$n := \text{floor}\left(\dfrac{E_s}{E_c}\right)$ $\boxed{n = 8}$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$ $\boxed{k = 0.3}$

$j := 1 - \dfrac{k}{3}$ $\boxed{j = 0.9}$

$d_c := cover + Bar_D(bar_{stirrup}) + \dfrac{Bar_D(Bar_{No\_pos})}{2}$ $\boxed{d_c = 3.69}$ in

$\boxed{Ms_{pos} = 1634}$ kip-ft

$f_s := \dfrac{Ms_{pos}}{As_{prov\_pos} \cdot j \cdot d_e} \cdot 12 \quad \leq 0.6\ f_y$ $\boxed{f_s = 36.24}$ ksi approx. = 0.6 f_y O.K.

The height of the section, h, is:

$h := cap_H \cdot 12$ $\boxed{h = 48}$ in

$\beta := 1 + \dfrac{d_c}{0.7 \cdot (h - d_c)}$ $\boxed{\beta = 1.12}$

$\gamma_e := 1.0$ for Class 1 exposure condition

$S_{max} := \dfrac{700\gamma_e}{\beta \cdot f_s} - 2 \cdot d_c$ $\boxed{S_{max} = 9.89}$ in

$\boxed{spa_{pos} = 4.33}$ in

Is $spa_{pos} \leq S_{max}$? $\boxed{check = \text{"OK"}}$

E13-2.6.2 Positive Moment Reinforcement Cut Off Location

Terminate the top row of bars where bottom row of reinforcement satisfies the moment diagram.

$spa' := spa_{pos}$ $\boxed{spa' = 4.33}$ in

$As' := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos1}$ $\boxed{As' = 9}$ in^2

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.85}$ (for $f'_c \leq 10.0$ ksi)

$$a' := \frac{As' \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c} \qquad \boxed{a' = 4.32} \quad \text{in}$$

$$d_{e'} := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - \frac{Bar_D(Bar_{No\_pos})}{2}$$

$$\boxed{d_{e'} = 44.31} \quad \text{in}$$

$$M_{n'} := As' \cdot f_y \cdot \left(d_{e'} - \frac{a'}{2}\right) \cdot \frac{1}{12} \qquad \boxed{M_{n'} = 1897} \quad \text{kip-ft}$$

$$M_{r'} := \phi_f \cdot M_{n'} \qquad \boxed{M_{r'} = 1707} \quad \text{kip-ft}$$

Based on the moment diagram, try locating the first cut off at $cut_{pos} := 10.7$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 1707}$ kip-ft

$\boxed{Mu_{cut1} = 1538}$ kip-ft

$\boxed{Ms_{cut1} = 1051}$ kip-ft

Is $Mu_{cut1} \leq M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{cut1} = 2045}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{cut1}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho' := \frac{As'}{b_w \cdot d_{e'}} \qquad \boxed{\rho' = 0.00484}$$

$$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n \qquad \boxed{k' = 0.24}$$

$j' := 1 - \frac{k'}{3}$ $\boxed{j' = 0.92}$

$\boxed{Ms_{cut1} = 1051}$ kip-ft

$f_{s'} := \frac{Ms_{cut1}}{As' \cdot j' \cdot d_{e'}} \cdot 12 \quad \leq 0.6\ f_y$ $\boxed{f_{s'} = 34.39}$ ksi $\leq 0.6\ f_y$ O.K.

$\boxed{\beta = 1.12}$

$\boxed{\gamma_e = 1}$

$S_{max'} := \frac{700\gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c$ $\boxed{S_{max'} = 10.81}$ in

$\boxed{spa' = 4.33}$ in

Is $spa' \leq S_{max'}$? $\boxed{check = \text{"OK"}}$

The bars shall be extended past this cut off point for a distance not less than the following, **LRFD [5.11.1.2.1]**:

$\boxed{d_{e'} = 44.31}$ in

$\boxed{15 \cdot Bar_D(Bar_{No\_pos}) = 16.92}$ in

$\boxed{\frac{col_{spa} \cdot 12}{20} = 10.95}$ in

$\boxed{BarExtend_{pos} = 44.31}$ in

The bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number $\boxed{\rightarrow 9}$ bar with spacing less than 6-inches, is:

$l_{d\_9} := 5.083$ ft

$\boxed{cut_{pos} + \frac{BarExtend_{pos}}{12} = 14.39}$

$\boxed{0.4 \cdot col_{spa} + l_{d\_9} = 12.38}$

Similar calculations show that the second layer bottom mat bars can also be terminated at a distance of 2.0 feet from the CL of the left column. At least one quarter of the bars shall be extended past the centerline of the support for continuous spans. Therefore, run the bottom layer bars to the end of the cap.

E13-2.6.3 Negative Moment Capacity at Face of Column

It is assumed that there will be one layer of negative moment reinforcement. Therefore the effective depth of the section at the pier is:

$cover = 2.5$ in

$bar_{stirrup} = 5$ (transverse bar size)

$Bar_D(bar_{stirrup}) = 0.63$ in (transverse bar diameter)

$Bar_{No\_neg} := 8$

$Bar_D(Bar_{No\_neg}) = 1.00$ in (Assumed bar size)

$$d_{e\_neg} := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - \frac{Bar_D(Bar_{No\_neg})}{2}$$

$d_{e\_neg} = 44.38$ in

For flexure in non-prestressed concrete, $\phi_f = 0.9$.

The width of the cap:

$b_w = 42$ in

$Mu_{neg} = -1174$ kip-ft

$$R_{u\_neg} := \frac{|Mu_{neg}| \cdot 12}{\phi_f \cdot b_w \cdot d_{e\_neg}^{\;2}}$$

$R_{u\_neg} = 0.1892$ ksi

$$\rho_{neg} := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_{u\_neg}}{0.85 \cdot f'_c}}\right)$$

$\rho_{neg} = 0.00326$

$$A_{s\_neg} := \rho_{neg} \cdot b_w \cdot d_{e\_neg}$$

$A_{s\_neg} = 6.08$ in^2

This requires $n_{bars\_neg} := 9$ bars. Check spacing requirements.

$$spa_{neg} := \frac{b_w - 2 \cdot (cover + Bar_D(bar_{stirrup})) - Bar_D(Bar_{No\_neg})}{n_{bars\_neg} - 1}$$

$spa_{neg} = 4.34$ in

$clear_{spa\_neg} := spa_{neg} - Bar_D(Bar_{No\_neg})$ $\boxed{clear_{spa\_neg} = 3.34}$ in

Is $spa_{min} \le clear_{spa\_neg}$? $\boxed{check = \text{"OK"}}$

$As_{prov\_neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg}$ $\boxed{As_{prov\_neg} = 7.07}$ in^2

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi)

$$a_{neg} := \frac{As_{prov\_neg} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a_{neg} = 3.39}$ in

$$Mn_{neg} := As_{prov\_neg} \cdot f_y \cdot \left(d_{e\_neg} - \frac{a_{neg}}{2}\right) \cdot \frac{1}{12}$$ $\boxed{Mn_{neg} = 1508}$ kip-ft

$Mr_{neg} := \phi_f \cdot Mn_{neg}$ $\boxed{Mr_{neg} = 1358}$ kip-ft

$\boxed{Mu_{neg} = 1174}$ kip-ft

Is $Mu_{neg} \le Mr_{neg}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg} = 1561}$ kip-ft

Is Mr_{neg} greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{neg}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho_{neg} := \frac{As_{prov\_neg}}{b_w \cdot d_{e\_neg}}$$ $\boxed{\rho_{neg} = 0.00379}$

$\boxed{n = 8}$

$$k_{neg} := \sqrt{(\rho_{neg} \cdot n)^2 + 2 \cdot \rho_{neg} \cdot n} - \rho_{neg} \cdot n$$ $\boxed{k_{neg} = 0.22}$

$$j_{neg} := 1 - \frac{k_{neg}}{3}$$ $\boxed{j_{neg} = 0.93}$

$$d_{c\_neg} := cover + Bar_D(bar_{stirrup}) + \frac{Bar_D(Bar_{No\_neg})}{2}$$ $\boxed{d_{c\_neg} = 3.63}$ in

$\boxed{Ms_{neg} = 844}$ kip-ft

$f_{s\_neg} := \dfrac{Ms_{neg}}{As_{prov\_neg} \cdot j_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \leq 0.6\ f_y$ $\qquad f_{s\_neg} = 34.8$ ksi $\leq 0.6\ f_y$ O.K.

The height of the section, h, is: $\boxed{h = 48}$ in

$\beta_{neg} := 1 + \dfrac{d_{c\_neg}}{0.7 \cdot (h - d_{c\_neg})}$ $\qquad \boxed{\beta_{neg} = 1.12}$

$\gamma_e := 1.0$ for Class 1 exposure condition

$S_{max\_neg} := \dfrac{700\gamma_e}{\beta_{neg} \cdot f_{s\_neg}} - 2 \cdot d_{c\_neg}$ $\qquad \boxed{S_{max\_neg} = 10.76}$ in

$\boxed{spa_{neg} = 4.34}$ in

Is $spa_{neg} \leq S_{max\_neg}$? $\qquad \boxed{check = \text{"OK"}}$

E13-2.6.4 Negative Moment Reinforcement Cut Off Location

Cut 4 bars where the remaining 5 bars satisfy the moment diagram.

$n_{bars\_neg'} := 5$

$spa'_{neg} := spa_{neg} \cdot 2$ $\qquad \boxed{spa'_{neg} = 8.69}$ in

$As'_{neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg'}$ $\qquad \boxed{As'_{neg} = 3.93}$ in^2

LRFD [5.7.2.2] $\qquad \alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi)

$a'_{neg} := \dfrac{As'_{neg} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$ $\qquad \boxed{a'_{neg} = 1.89}$ in

$\boxed{d_{e\_neg} = 44.38}$ in

$M_{n'\_neg} := As'_{neg} \cdot f_y \cdot \left(d_{e\_neg} - \dfrac{a'_{neg}}{2}\right) \cdot \dfrac{1}{12}$ $\qquad \boxed{M_{n'\_neg} = 853}$ kip-ft

$M_{r'\_neg} := \phi_f \cdot M_{n'\_neg}$ $\qquad \boxed{M_{r'\_neg} = 768}$ kip-ft

Based on the moment diagram, try locating the cut off at $cut_{neg} := 15.3$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'\_neg} = 768}$ kip-ft

$\boxed{Mu_{neg\_cut} = 577}$ kip-ft

$\boxed{Ms_{neg\_cut} = 381}$ kip-ft

Is $Mu_{neg\_cut} \le M_{r'\_neg}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg\_cut} = 767}$ kip-ft

Is $M_{r'\_neg}$ greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{neg\_cut}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho'_{neg} := \frac{As'_{neg}}{b_w \cdot d_{e\_neg}} \qquad \boxed{\rho'_{neg} = 0.00211}$$

$$k'_{neg} := \sqrt{(\rho'_{neg} \cdot n)^2 + 2 \cdot \rho'_{neg} \cdot n} - \rho'_{neg} \cdot n \qquad \boxed{k'_{neg} = 0.17}$$

$$j'_{neg} := 1 - \frac{k'_{neg}}{3} \qquad \boxed{j'_{neg} = 0.94}$$

$\boxed{Ms_{neg\_cut} = 381}$ kip-ft

$$f_{s'\_neg} := \frac{Ms_{neg\_cut}}{As'_{neg} \cdot j'_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \le 0.6\, f_y \qquad \boxed{f_{s'\_neg} = 27.79} \text{ ksi} \quad \le 0.6\, f_y \text{ O.K.}$$

$\boxed{\beta_{neg} = 1.12}$

$\boxed{\gamma_e = 1}$

$$S_{max'\_neg} := \frac{700 \gamma_e}{\beta_{neg} \cdot f_{s'\_neg}} - 2 \cdot d_{c\_neg} \qquad \boxed{S_{max'\_neg} = 15.30} \text{ in}$$

$\boxed{spa'_{neg} = 8.69}$ in

Is $spa'_{neg} \le S_{max'\_neg}$? $\boxed{check = \text{"OK"}}$

The bars shall be extended past this cut off point for a distance not less than the following, **LRFD [5.11.1.2.3]**:

$\boxed{d_{e\_neg} = 44.38}$ in

$\boxed{12 \cdot Bar_D(Bar_{No\_neg}) = 12}$ in

$\boxed{\dfrac{(col_{spa} - col_w) \cdot 12}{16} = 10.69}$ in

$\boxed{BarExtend_{neg} = 44.38}$ in

These bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number $\boxed{\rightarrow 8}$ "top" bar with spacing greater than 6-inches, is:

$l_{d\_8} := 3.25$ ft

The cut off location is determined by the following:

$\boxed{cut_{neg} - \dfrac{BarExtend_{neg}}{12} = 11.6}$ ft

$\boxed{col_{spa} - \dfrac{col_w}{2} - l_{d\_8} = 13}$ ft

Therefore, the cut off location is located at the following distance from the CL of the left column:

$\boxed{cutoff_{location} = 11.6}$ ft

By inspection, the remaining top mat reinforcement is adequate over the exterior columns. The inside face of the exterior column is located at:

$col_{face} := \dfrac{col_w}{2} \cdot \dfrac{1}{col_{spa}}$

$\boxed{col_{face} = 0.11}$ % along cap

$\boxed{Mu_{negative}(col_{face}) = -378.37}$ kip-ft

$\boxed{Ms_{negative}(col_{face}) = -229.74}$ kip-ft

E13-2.6.5 Shear Capacity at Face of Center Column

$\boxed{Vu = 978.82}$ kips

The Factored Shear Resistance, V_r

$$V_r = \phi_v (V_n)$$

$$\phi_v := 0.9$$

V_n is determined as the lesser of the following equations, **LRFD [5.8.3.3]**:

$$V_{n1} = V_c + V_s + V_p$$

$$V_{n2} = 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$$

V_c , the shear resistance due to concrete (kip), is calculated as follows:

$$V_c = 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$$

Where:

b_v = effective web width (in) taken as the minimum section width within the depth d_v

d_v = effective shear depth (in), the distance, measured perpendicular to the neutral axis between the resultants of the tensile and compressive force due to flexure. It need not be taken less than the greater of $0.9d_e$ or $0.72h$

$b_v := cap_W \cdot 12$ $\boxed{b_v = 42}$ in

$\boxed{d_{e\_neg} = 44.38}$ in

$\boxed{a_{neg} = 3.39}$ in

$d_{v\_neg} := d_{e\_neg} - \frac{a_{neg}}{2}$ $\boxed{d_{v\_neg} = 42.68}$ in

$\boxed{0.9 \cdot d_{e\_neg} = 39.94}$ in

$\boxed{h = 48}$ in $\boxed{0.72 \cdot h = 34.56}$ in

Therefore, use $\boxed{d_v = 42.68}$ in for V_c calculation.

$\beta := 2.0$ Factor indicating ability of diagonally cracked concrete to transmit tension. For nonprestressed sections, β = 2.0, **LRFD [5.8.3.4.1]**.

$\lambda := 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_c := 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_c = 211.94}$ kips

V_s , the shear resistance due to steel (kips), is calculated as follows:

$$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s}$$

Where:

s = spacing of stirrups (in)

θ = angle of inclination of diagonal compressive stresses (deg)

α = angle of inclination of transverse reinforcement to longitudinal axis (deg)

$s := 5$ in

$\theta := 45deg$ for non prestress members

$\alpha := 90deg$ for vertical stirrups

A_v = (# of stirrup legs)(area of stirrup)

$\boxed{bar_{stirrup} = 5}$

StirrupConfig := "Triple"

$\boxed{stirrup_{legs} = 6}$

$A_v := stirrup_{legs} \cdot \left(Bar_A\left(bar_{stirrup}\right)\right)$ $\boxed{A_v = 1.84}$ in^2

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s}$$ $\boxed{V_s = 942.74}$ kips

V_p , the component of the effective prestressing force in the direction of the applied shear:

$V_p := 0$ for non prestressed members

V_n is the lesser of:

$V_{n1} := V_c + V_s + V_p$ $\boxed{V_{n1} = 1154.67}$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $\boxed{V_{n2} = 1568.41}$ kips

Therefore, use: $\boxed{V_n = 1154.67}$ kips

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 1039.2}$ kips

$\boxed{Vu = 978.82}$ kips

Is $Vu \le V_r$? $\boxed{check = \text{"OK"}}$

Check the Minimum Transverse Reinforcement, **LRFD [5.8.2.5]**

Required area of transverse steel:

$\lambda := 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$Av_{min} := 0.0316 \cdot \frac{\lambda \sqrt{f'_c} \cdot b_v \cdot s}{f_y}$$

$\boxed{Av_{min} = 0.21}$ in^2

$\boxed{A_v = 1.84}$ in^2

Is $Av_{min} \le A_v$ (provided area of steel)? $\boxed{check = \text{"OK"}}$

Check the Maximum Spacing of the Transverse Reinforcement, **LRFD [5.8.2.7]**

If $v_u < 0.125f'_c$, then: $s_{max} := 0.8 \cdot d_v \le 24$ in

If v_u > or = $0.125f'_c$, then: $s_{max} := 0.4 \cdot d_v \le 12$ in

The shear stress on the concrete, v_u, is taken to be:

$$v_u := \frac{Vu}{\phi_v \cdot b_v \cdot d_v}$$

$\boxed{v_u = 0.61}$ ksi

$\boxed{0.125 \cdot f'_c = 0.44}$ ksi

$\boxed{s_{max} = 12}$ in

$\boxed{s = 5}$ in

Is the spacing provided $s \le s_{max}$? $\boxed{check = \text{"OK"}}$

Similar calculations are used to determine the required stirrup spacing for the remainder of the cap.

$\boxed{s_2 = 12}$	in	$\boxed{s_3 = 6}$	in
$\boxed{StirrupConfig_2 = \text{"Double"}}$		$\boxed{StirrupConfig_3 = \text{"Double"}}$	
$\boxed{Vu2 = 276}$	kips	$\boxed{Vu3 = 560}$	kips
$\boxed{V_{r\_2} = 408.94}$	kips	$\boxed{V_{r\_3} = 627.13}$	kips

It should be noted that the required stirrup spacing is typically provided for a distance equal to the cap depth past the CL of the girder. Consideration should also be given to minimize the number of stirrup spacing changes where practical. These procedures result in additional capacity in the pier cap that is often beneficial for potential future rehabilitation work on the structure.

E13-2.6.6 Temperature and Shrinkage Steel

Temperature and shrinkage steel shall be provided on each face and in each direction as calculated below. **LRFD [5.10.8]**

$\boxed{cap_W = 3.5}$ ft

$\boxed{cap_H = 4}$ ft

$b := cap_W \cdot 12$ $\boxed{b = 42}$ in

$\boxed{h = 48}$ in

$As_{ts} := \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y}$ $\boxed{As_{ts} = 0.24}$ in^2/ft in each face

Is the area required As_{ts} between 0.11 and 0.60 in^2 per foot? $\boxed{check = \text{"OK"}}$

Use number 5 bars at one foot spacing: $\boxed{Bar_A(5) = 0.31}$ in^2/ft in each face

E13-2.6.7 Skin Reinforcement

If the effective depth, d_e, of the reinforced concrete member exceeds 3 ft., longitudinal skin reinforcement is uniformly distributed along both side faces of the component for a distance of de/2 nearest the flexural tension reinforcement, **LRFD [5.7.3.4]**. The area of skin reinforcement (in^2/ft of height) on each side of the face is required to satisfy:

$A_{sk} \geq 0.012(d_e - 30)$ and $A_{sk} \cdot \left(\frac{d_e}{2 \cdot 12}\right)$ need not exceed $(A_s / 4)$

Where: (For positive moment region)

A_{sk} = area of skin reinforcement (in^2/ft)

A_s = area of tensile reinforcement (in^2) $\boxed{A_s = 13.28}$ in^2

d_e = flexural depth taken as the distance from the compression face to the centroid of the steel, positive moment region (in) $\boxed{d_e = 42.87}$ in

$A_{sk1} := 0.012 \cdot (d_e - 30)$ $\boxed{A_{sk1} = 0.15}$ in^2/ft

$A_{sk1} := A_{sk1} \cdot \left(\frac{d_e}{2 \cdot 12}\right)$ $\boxed{A_{sk1} = 0.28}$ in^2

$A_{sk2} := \frac{A_s}{4}$ $\boxed{A_{sk2} = 3.32}$ in^2

$A_{face} := \min(A_{sk1}, A_{sk2})$ (area req'd. per face within de/2 from tension reinf.) $\boxed{A_{face} = 0.28}$ in^2

$spa\_max_{sk} := \min\left(\frac{d_e}{6}, 12\right)$ $\boxed{spa\_max_{sk} = 7.15}$ in

Use number 5 bars at 6" spacing:
(provides 2 bars within de/2 from tension reinf.) $\boxed{Bar_A(5) \cdot 2 = 0.61}$ in^2 > A_{face}

Preceding calculations looked at skin reinforcement requirements in the positive moment region. For the negative moment region, #5 bars at 6" will also meet its requirements.

E13-2.7 Reinforcement Summary

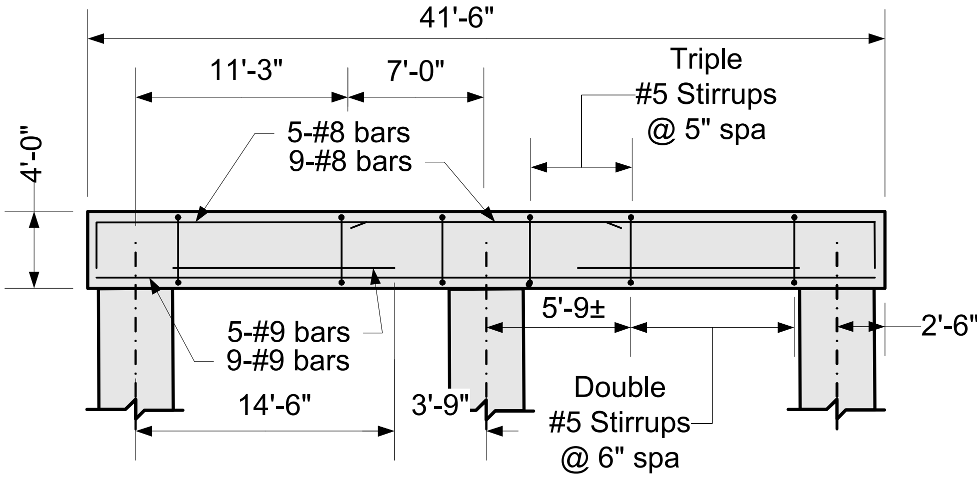


Figure E13-2.7-1
Cap Reinforcement - Elevation View

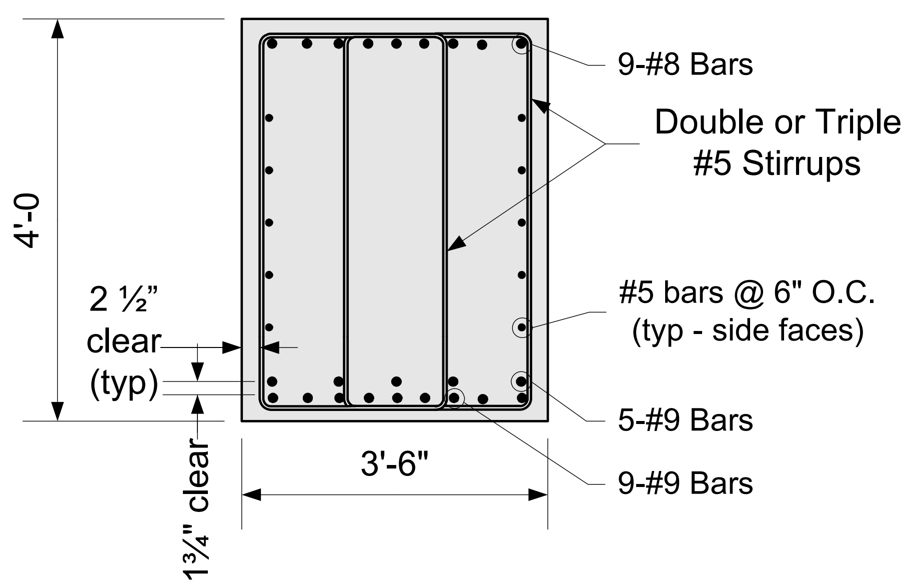


Figure E13-2.7-2
Cap Reinforcement - Section View

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Table of Contents

14.1 Introduction

Retaining walls are used to provide lateral resistance for a mass of earth or other material to accommodate a transportation facility. These walls are used in a variety of applications including right-of-way restrictions, protection of existing structures that must remain in place, grade separations, new highway embankment construction, roadway widening, stabilization of slopes, protection of environmentally sensitive areas, staging, and temporary support including excavation or underwater construction support, etc.

Several types of retaining wall systems are available to retain earth and meet specific project requirements. Many of these wall systems are proprietary wall systems while others are non-proprietary or design-build in Wisconsin. The wall selection criteria and design policies presented in this chapter are to ensure consistency of standards and applications used throughout WisDOT projects.

WisDOT policy item:

Retaining walls (such as MSE walls with precast concrete panel facing) that are susceptible to damage from vehicular impact shall be protected by a roadway barrier.

14.1.1 Wall Development Process

Overall, the wall development process requires an iterative collaboration between WisDOT Regions, Structures Design Section, Geotechnical Engineering Unit and WisDOT Consultants.

Retaining wall development is described in Section 11-55-5 of the *Facilities Development Manual*. WisDOT Regional staff determines the need for permanent retaining walls on highway projects. A wall number is assigned as per criteria discussed in 14.1.1.1 of this chapter. The Regional staff prepares a Structures Survey Report (SSR) that includes a preliminary evaluation of wall type, location, and height including a preliminary layout plan.

Based on the SSR, a Geotechnical site investigation (see Chapter 10 – Geotechnical Investigation) may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Geotechnical Engineering Unit can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results. These Geotechnical recommendations are presented in a Site Investigation Report.

The SSR is sent to the wall designer (Structures Design Section or WisDOT's Consultant) for wall selection, design and contract plan preparation. Based on the wall selection criteria discussed in 14.3, either a proprietary or a non-proprietary wall system is selected.

Proprietary walls, as defined in 14.2, are pre-approved by the WisDOT's Bureau of Structures. Preapproval process for the proprietary walls is explained in 14.16. The structural design, internal and final external stability of proprietary wall systems are the responsibility of the supplier/contractor. The design and shop drawing computations of the proprietary wall systems are also reviewed by the Bureau of Structures in accordance with the plans and special provisions. The preliminary external stability, overall stability and settlement computations of these walls are performed by the Geotechnical Engineering Unit or the WisDOT's Consultant in the project design phase. Design and shop drawings must be accepted by the Bureau of Structures prior to start of the construction. Design of all temporary walls is the responsibility of the contractor.

Non-proprietary retaining walls are designed by WisDOT or its Consultant. The internal stability and the structural design of such walls are performed by the Structures Design Section or WisDOT's Consultant. The external and overall stability is performed by the Geotechnical Engineering Unit or Geotechnical Engineer of record.

The final contract plans of retaining walls include final plans, details, special provisions, contract requirements, and cost estimate for construction. The Subsurface Exploration sheet depicting the soil borings is part of the final contract plans.

The wall types and wall selection criteria to be used in wall selection are discussed in 14.2 and 14.3 of this chapter respectively. General design concepts of a retaining wall system are discussed in 14.4. Design criteria for specific wall systems are discussed in sections 14.5 thru 14.11. The plan preparation process is briefly described in Chapter 2 – General and Chapter 6 – Plan Preparation. The contract documents and contract requirements are discussed in 14.14 and 14.15 respectively.

For further information related to wall selection, design, approval process, pre-approval and review of proprietary wall systems please contact Structures Design Section of the Bureau of Structures at 608-266-8489. For questions pertaining to geotechnical analyses and geotechnical investigations please contact the Geotechnical Engineering Unit at 608-246-7940.

14.1.1.1 Wall Numbering System

Permanent retaining walls that are designed for a design life of 75 years or more should be identified by a wall number, R-XX-XXX, as assigned by the Region unless otherwise specified below. For a continuous wall consisting of various wall types, the numbering system should include unit numbers so that the numbering appears as R-XX-XXX-001, R-XX-XXX-002, and so on. The first two digits represent the county the wall is located in and the next set(s) of digits represent the undivided wall.

Retaining walls whose height exceeds the following criteria require R numbers:

- <u>Proprietary retaining walls</u> (e.g., modular block MSE walls)
    - MSE walls having a maximum height of less than 5.5 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be "minor retaining walls" and do not require an R number. Refer to *FDM 11-55-5.2* for more information.
    - Modular block gravity walls having a maximum height of less than 4.0 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be "minor

retaining walls" and do not require an R number. Refer to *FDM 11-55-5.2* for more information.

- Non-proprietary walls (e.g., sheet pile walls):
    - Walls having an exposed height of less than 5.5 ft. measured from the plan ground line to top of wall may require an R number based on specific project features. Designer to contact the Bureau of Structures region liaison for more information.

Cast-in-place walls being utilized strictly as bridge abutment or box culvert wings do not require R numbers as they are considered part of the structure.

14.2 Wall Types

Retaining walls can be divided into many categories as discussed below.

Conventional Walls

Retaining walls can be divided into gravity, semi-gravity, and non-gravity cantilever or anchored walls. A brief description of these walls is presented in 14.2.1 and 14.2.2 respectively.

Miscellaneous types of walls including multi-tiered walls, and hybrid or composite walls are also used by combining the wall types mentioned in the previous paragraph. These walls are used only under special project requirements. These walls are briefly discussed in 14.2.3, but the design requirements of these walls will not be presented in this chapter. In addition, some walls are also used for temporary shoring and discussed briefly in 14.2.4.

Permanent or Temporary Walls

All walls can be divided into permanent or temporary walls, depending on project application. Permanent walls have a typical designed life of 75 years. The temporary walls are designed for a service life of 3 years, or the intended project duration, whichever is greater. Temporary wall systems have less restrictive requirements for construction, material and aesthetics.

Fill Walls or Cut Walls

A retaining wall can also be classified as a fill wall, or a cut wall. This description is based on the nature of the earthwork required to construct the wall. If the roadway cross-sections (which include the wall) indicate that existing earth/soil must be removed (excavated) to install the wall, it is considered a ‘cut’ wall. If the roadway cross-sections indicate that earth fill will be placed behind the wall, with little excavation, the wall is considered a ‘fill’ wall. Sometimes wall construction requires nearly equal combinations of earth excavation and earth fill, leading to the nomenclature of a ‘cut/fill’ wall.

Bottom-up or Top-down Constructed Walls

This wall classification method refers to the method in which a wall is constructed. If a wall is constructed from the bottom of the wall, upward to the top, it is considered a bottom-up type of wall. Examples of this include CIP cantilever, MSE and modular block walls. Bottom-up walls are generally the most cost effective type. If a wall is constructed downward, from the top of the wall to the bottom, it is considered a top-down type of wall. This generally requires the insertion of some type of wall support member below the existing ground, and then excavation in front of the wall to the bottom of the exposed face. Examples of this include soil nail, soldier pile, cantilever sheet pile and anchored sheet pile walls. These walls are generally used when excavation room is limited.

Proprietary or Non-Proprietary

Some retaining walls have prefabricated modules or components that are proprietary in nature. Based on the use of proprietary components, walls can be divided into the categories of proprietary and non-proprietary wall systems as defined in 14.1.1.

A proprietary retaining wall system is considered as a patented or trademarked retaining wall system or a wall system comprised of elements/components that are protected by a trade name, brand name, or patent and are designed and supported by the manufacturer. MSE walls, modular block gravity walls, bin, and crib walls are considered proprietary walls because these walls have components which are either patented or have trademarks.

Proprietary walls require preapproval and appropriate special provisions. The preapproval requirements are discussed in 14.16 of this chapter. Proprietary walls also have special design requirements for the structural components, and are discussed in further detail within each specific wall design section. Most MSE, modular block, bin or crib walls require pre-approval and/or special provisions.

A non-proprietary retaining wall is fully designed and detailed by the designer or may be design-build. A non-proprietary retaining wall system may contain proprietary elements or components as well as non-proprietary elements and components. CIP cantilever walls, rock walls, soil nail walls and non-gravity walls fall under this category.

Wall classification is shown in Table 14.2-1 and is based on wall type, project function category, and method of construction.

14.2.1 Gravity Walls

Gravity walls are considered externally stabilized walls as these walls use self weight to resist lateral pressures due to earth and water. Gravity walls are generally subdivided into mass gravity, semi-gravity, modular gravity, mechanically stabilized reinforced earth (MSE), and in-situ reinforced earth wall (soil nailing) categories. A schematic diagram of the various types of gravity walls is included in Figure 14.2-1.

14.2.1.1 Mass Gravity Walls

A mass gravity wall is an externally stabilized, cast-in-place rigid gravity wall, generally trapezoidal in shape. The construction of these walls requires a large quantity of materials so these are rarely used except for low height walls less than 8.0 feet. These walls mainly rely on self-weight to resist external pressures and their construction is staged as bottom up construction, mostly in fill or cut/fill situations.

14.2.1.2 Semi-Gravity Walls

Semi-gravity walls resist external forces by the combined action of self-weight, weight of soil above footing and the flexural resistance of the wall components. A cast-in-place (CIP) concrete cantilever wall is an example and consists of a reinforced concrete stem and a base footing. These walls are non-proprietary.

Cantilever walls are best suited for use in areas exhibiting good bearing material. When bearing or settlement is a problem, these walls can be founded on piles or foundation improvement may be necessary. The use of piles significantly increases the cost of these walls. Walls exceeding 28 feet in height are provided with counter-forts or buttress slabs. Construction of these walls is staged as bottom-up construction and mostly constructed in fill situations. Cantilever walls are more suited where MSE walls are not feasible, although these walls are generally costlier than MSE walls.

14.2.1.3 Modular Gravity Walls

Modular walls are also known as externally stabilized gravity walls as these walls resist external forces by utilizing self-weight. Modular walls have prefabricated modules/components which are considered proprietary. The construction is bottom-up construction mostly used in fill situations.

14.2.1.3.1 Modular Block Gravity Walls

Modular block concrete facings are used without soil reinforcement to function as an externally stabilized gravity wall. The modular blocks are prefabricated dry cast or wet cast concrete blocks and the blocks are stacked vertically or slightly battered to resist external forces. The concrete blocks are either solid concrete or hollow core concrete blocks. The hollow core concrete blocks are filled with crushed aggregates or sand. Modular block gravity walls are limited to a maximum design height of 8 feet under optimum site geometry and soils conditions, but site conditions generally dictate the need for MSE walls when design heights are greater than 5.5 feet. Walls with a maximum height of less than 4 feet are deemed as “minor retaining walls” and do not require an R number. Refer to *FDM 11-55-5.2* for more information. The modular blocks are proprietary and vary in sizes.

14.2.1.3.2 Prefabricated Bin, Crib and Gabion Walls

<u>Bin Walls</u>: Concrete and metal bin walls are built of adjoining open or closed faced bins and then filled with soil/rocks. Each metal bin is comprised of individual members bolted together. The concrete bin wall is comprised of prefabricated interlocking concrete modules. These wall systems are proprietary wall systems.

<u>Crib Walls</u>: Crib walls are constructed of interlocking prefabricated units of reinforced or unreinforced concrete or timber elements. Each crib is comprised of longitudinal and transverse members. Each unit is filled with free draining material. These wall systems are proprietary wall systems.

<u>Gabion Walls</u>**:** Gabion walls are constructed of steel wire baskets filled with selected rock fragments and tied together. Gabions walls are flexible, free draining and easy to construct. These wall systems are proprietary wall systems. Maximum heights are normally less than 21 feet. These walls are desirable where equipment access is limited. The wires used for constructing gabions baskets must be designed with adequate corrosion protection.

14.2.1.4 Rock Walls

Rock walls are also known as 'Rockery Walls'. These types of gravity walls are built by stacking locally available large stones or boulders into a trapezoid shape. These walls are highly flexible and height of these walls is generally limited to approximately 8.0 feet. A layer of gravel and geotextile is commonly used between the stones and the retained soil. These walls can be designed using the *FHWA Rockery Design and Construction Guideline*.

14.2.1.5 Mechanically Stabilized Earth (MSE) Walls:

Mechanically Stabilized Earth (MSE) walls include a selected soil mass reinforced with metallic or geosynthetic reinforcement. The soil reinforcement is connected to a facing element to prevent the reinforced soil from sloughing. Construction of these walls is staged as bottom-up construction. These can be constructed in cut and fill situations, but are better suited to fill sites. MSE walls are normally used for wall heights between 10 to 40 feet. A brief description of various types of MSE walls is given below:

<u>Precast Concrete Panel MSE Walls</u>: These types of walls employ a metallic strip or wire grid reinforcement connected to precast concrete panels to reinforce a selected soil mass. The concrete panels are usually 5'x5' or 5'x10' size panels. These walls are proprietary wall systems.

<u>Modular Block Facing MSE Wall</u>: Prefabricated modular concrete block walls consist of almost vertically stacked concrete modular blocks and the soil reinforcement is secured between the blocks at predetermined levels. Metallic strips or geogrids are generally used as soil reinforcement to reinforce the selected soil mass. Concrete blocks are either solid or hollow core blocks, and must meet freeze/thaw requirements. The hollow core blocks are filled with aggregates or sand. These types of walls are proprietary wall systems.

<u>Geotextile/Geogrids/Welded Wire Faced MSE Walls</u>: These types of MSE walls consist of compacted soil layers reinforced with continuous or semi-continuous geotextile, geogrid or welded wire around the overlying reinforcement. The wall facing is formed by wrapping each layer of reinforcement around the overlying layer of backfill and re-embedding the free end into the backfill. These types of walls are used for temporary or permanent applications. Permanent facings include shotcrete, gunite, galvanized welded wire mesh, cast-in-place concrete or prefabricated concrete panels.

14.2.1.6 Soil Nail Walls

Soil nail walls are internally stabilized cut walls that use in-situ reinforcement for resisting earth pressures. The large diameter rebars (generally #10 or greater) are typically used for the reinforcement. The construction of soil nail walls is staged top-down and soil nails are installed after each stage of excavation. Shotcrete can be applied as a facing. The facing of a soil nail wall is typically covered with vertical drainage strips located over the nail then covered with shotcrete. Soil nail walls are used for temporary or permanent construction. Specialty contractors are required when constructing these walls. Soil nail walls have been installed to heights of 60.0 feet or more but there have only been a limited number of soil nail walls constructed on WisDOT projects.

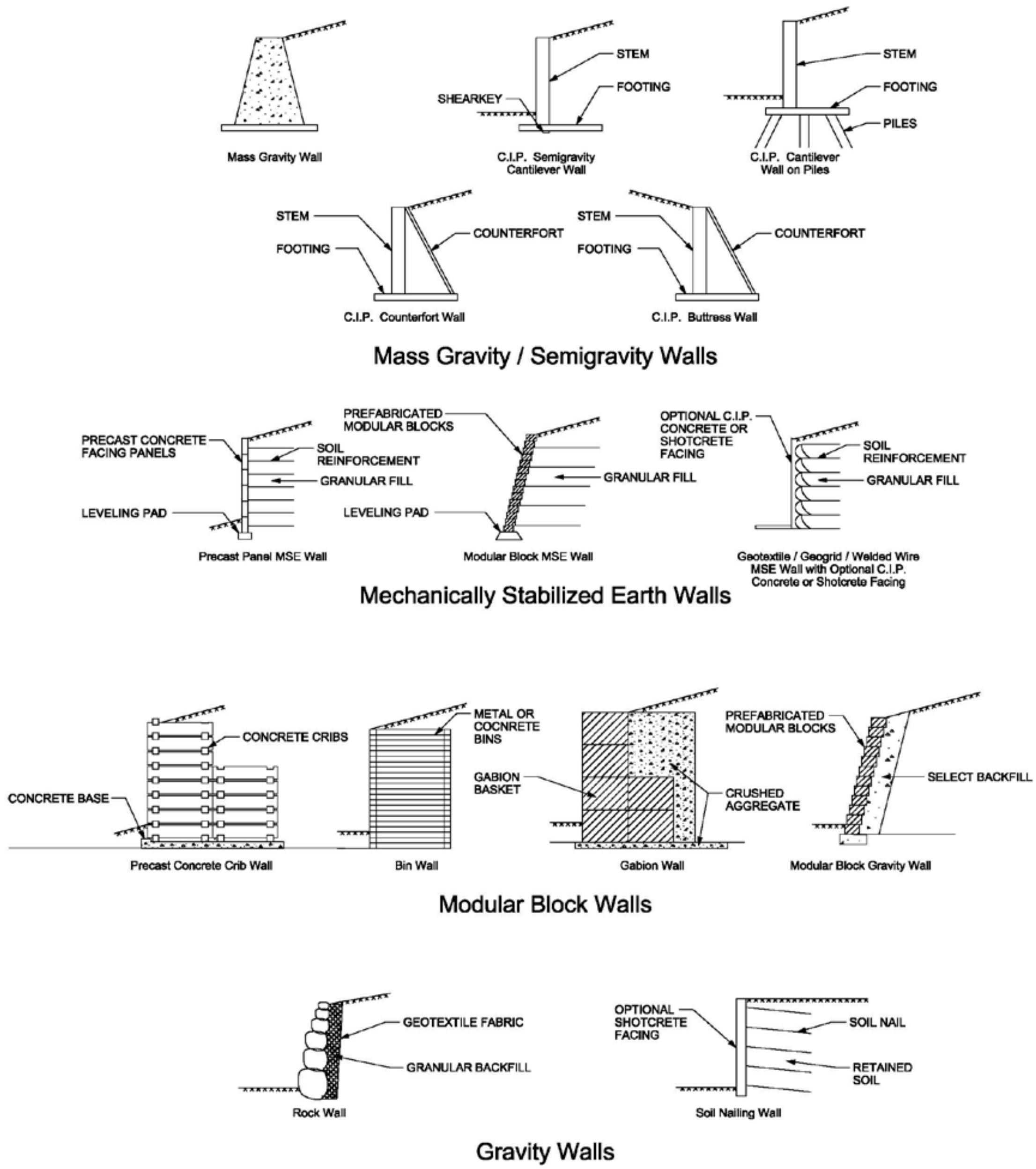


Figure 14.2-1
Gravity Walls

14.2.2 Non-Gravity Walls

Non-gravity walls are classified into cantilever and anchored wall categories. These walls are considered as externally stabilized walls and generally used in cut situations. The walls include sheet pile, soldier pile, tangent and secant pile type with or without anchors. Figure 14.2-2 shows common types of non-gravity walls.

14.2.2.1 Cantilever Walls

These types of walls derive lateral resistance through embedment of vertical elements into natural ground and the flexure resistance of the structural members. They are used where excavation support is needed in shallow cut situations.

Cantilever Sheet Pile Walls: Cantilever sheet pile walls consist of interlocking steel panels, driven into the ground to form a continuous sheet pile wall. The sheet piles resist the lateral earth pressure utilizing the passive resistance in front of the wall and the flexural resistance of the sheet pile. Most sheet pile walls are less than 15 feet in height.

Soldier Pile Walls: A soldier pile wall derives lateral resistance and moment capacity through embedment of vertical members (soldier piles) into natural ground usually in cut situations. The vertical elements (usually H piles) may be drilled or driven steel or concrete members. The soil behind the wall is retained by lagging. The lagging may be steel, wood, or concrete. For permanent walls, wall facings are usually constructed of either cast-in-place concrete or precast concrete panels (prestressed, if needed) that extend between vertical elements. Solider pile walls that use precast panels and H piles are also known as post-and-panel walls. Soldier pile walls can also be constructed from the bottom-up. These walls should be considered when minimizing disturbance to the site is critical, such as environmental and/or construction procedures. Soldier pile walls are also suitable for sites where rock is encountered near the surface, since holes for the piles can be drilled/prebored into the rock.

Tangent and Secant Pile Walls: A tangent pile wall consists of a single row of drilled shafts (bored piles) installed in the ground. Each pile touches the adjacent pile tangentially. The concrete piles are reinforced using a single steel beam or a steel reinforcement cage. A secant wall, similar to a tangent pile wall, consists of overlapping adjacent piles. All piles generally contain reinforcement, although alternating reinforced piles may be necessary. Secant and tangent wall systems are used to hold earth and water where water tightness is important, and lowering of the water table is not desirable. To improve wall water tightness, additional details can used to minimize water seepage.

14.2.2.2 Anchored Walls

Anchored walls are externally stabilized non-gravity cut walls. Anchored walls are essentially the same as cantilever walls except that these walls utilize anchors (tiebacks) to extend the wall heights beyond the design limit of the cantilever walls. These walls require less toe embedment than cantilever walls.

These walls derive lateral resistance by embedment of vertical wall elements into firm ground and by anchorages. Most commonly used anchored walls are anchored sheet pile walls and soldier pile walls. Tangent and secant walls can also be anchored with tie backs and used as anchored walls. The anchors can be attached to the walls by tie rods, bars or wire tendons. The anchoring device is generally a deadman, screw-type, or grouted tieback anchor. Anchored walls can be built to significant heights using multiple rows of anchors.

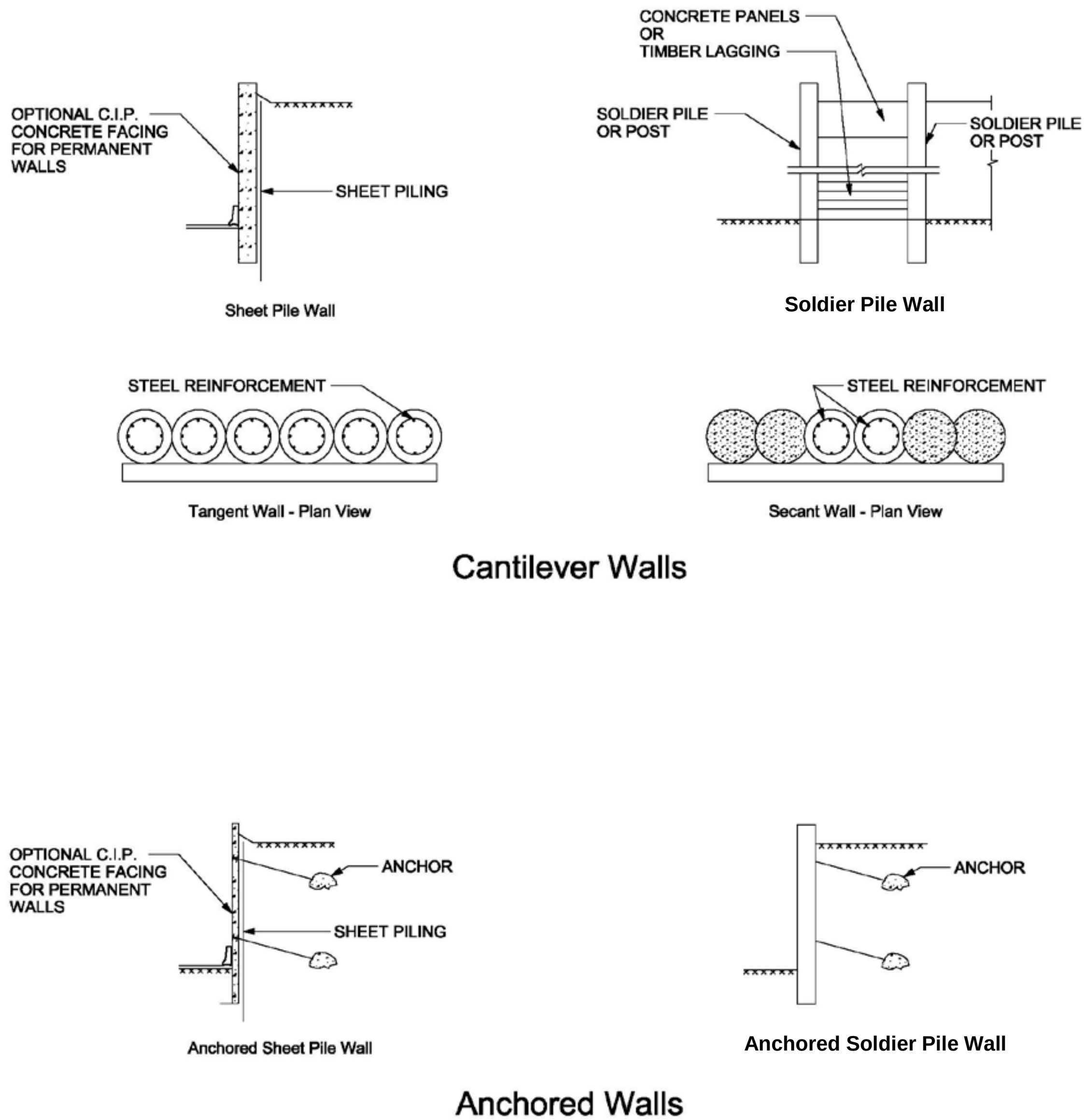


Figure 14.2-2
Non-Gravity Walls

14.2.3 Tiered and Hybrid Wall Systems

A tiered wall system is a series of two or more walls, with each wall set back from the underlying walls. The upper wall exerts an additional surcharge on the lower lying wall and requires special design attention. The design of these walls has not been discussed in this chapter. Hybrids wall systems combine wall components from two or more different wall systems and provide an alternative to a single type of wall used in cut or fill locations. These types of walls require special design attention as components of these walls require different magnitudes of deformation to develop loading resistance. The design of such walls will be on a case-by-case basis, and is not discussed in this chapter.

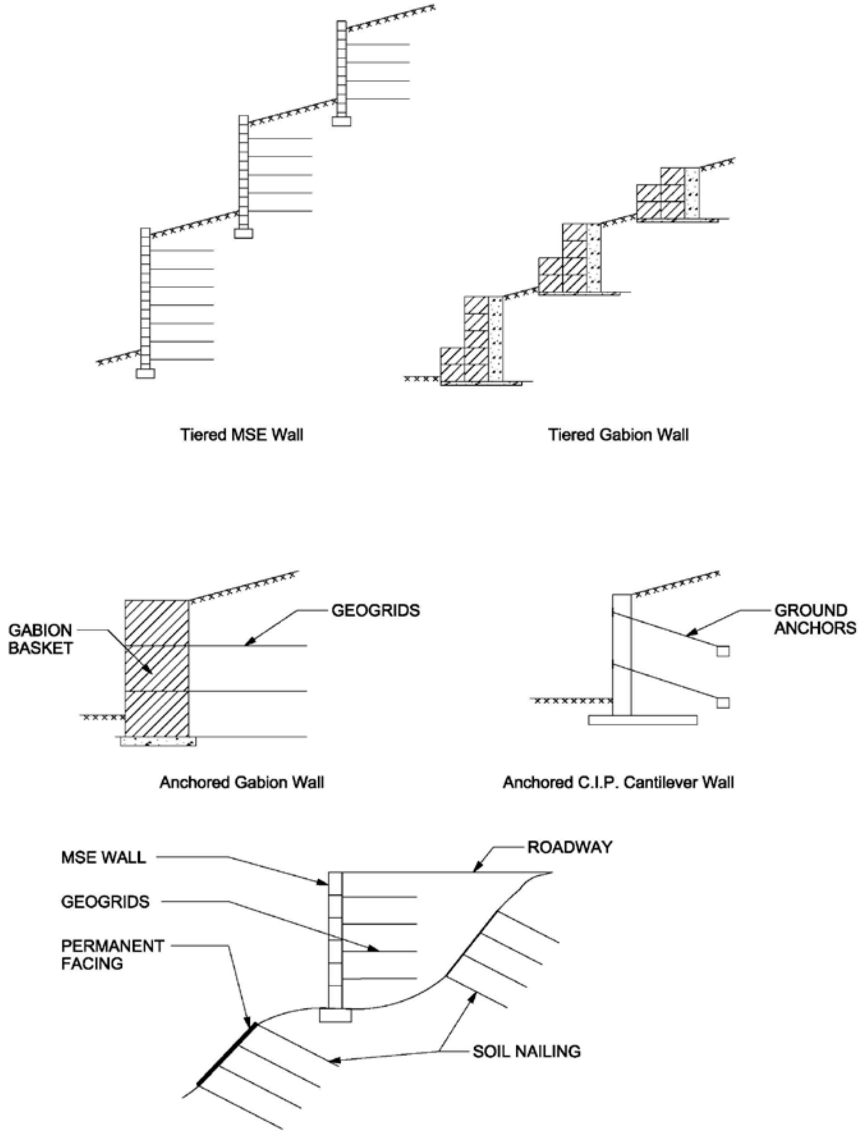
Some examples of tiered and hybrid walls systems are shown in Figure 14.2-3.

14.2.4 Temporary Shoring

Temporary shoring is used to protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Shoring should not be required nor paid for when used primarily for the convenience of the contractor. Temporary shoring is designed by the contractor and may consist of a wall system, or some other type of support. MSE walls with flexible facings and sheet pile walls are commonly used for temporary shoring.

14.2.5 Wall Classification Chart

A wall classification chart has been developed and shown as Table 14.2-1.



<u>Figure 14.2-3</u>
Tiered & Hybrid Wall Systems

Wall Category	Wall Sub-Category	Wall Type	Typical Construction Concept	Proprietary
Gravity	Mass Gravity	CIP Concrete Gravity	Bottom Up (Fill)	No
	Semi-Gravity	CIP Concrete Cantilever	Bottom Up (Fill)	No
	Reinforced Earth	MSE Walls: • Precast Panels • Modular Blocks • Geogrid/ Geo-textile/Wire- Faced	Bottom Up (Fill)	Yes
	Modular Gravity	Modular Blocks, Gabion, Bin, Crib	Bottom Up (Fill)	Yes
	In-situ Reinforced	Soil Nailing	Top Down (Cut)	No
Non-Gravity	Cantilever	Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut) /Bottom Up (Fill)	No
	Anchored	Anchored Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut)	No

Table 14.2-1
Wall Classification

14.3 Wall Selection Criteria

14.3.1 General

The objective of selecting a wall system is to determine an appropriate wall system that is practical to construct, structurally sound, economic, aesthetically pleasing, environmentally consistent with the surroundings, and has minimal maintenance problems.

With the development of many new wall systems, designers have the choice of selecting many feasible wall systems that can be constructed on a given highway project. Designers are encouraged to evaluate several feasible wall systems for a particular project where wall systems can be economically constructed. After consideration of various wall types, a single type should be selected for final analyses and design. Wall designers must consider the general design concepts described in section 14.4 and specific wall design requirements described in 14.5 thru 14.11 of this chapter, and key wall selection factors discussed in this section.

In general, selection of a wall system should include, but not limited to the key factors described in this section for consideration when generating a list of acceptable retaining wall systems for a given site.

14.3.1.1 Project Category

The designer must determine if the wall system is permanent or temporary.

14.3.1.2 Cut vs. Fill Application

Due to construction techniques and base width requirements for stability, some wall types are better suited for cut sections where as others are suited for fill or fill/cut situations. The key considerations are the amount of excavation or shoring, overall wall height, proximity of wall to other structures, and right-of-way width available. The site geometry should be evaluated to define site constraints. These constraints will generally dictate if fill, fill/cut or cut walls are required.

Cut Walls

Cut walls are generally constructed from the top down and used for both temporary and permanent applications. Cantilever sheet pile walls are suitable for shallower cuts. If a deeper cut is required to be retained, a key question is to determine the availability of right-of-way (ROW). Subsurface conditions such as shallow bedrock also enter into considerations of cut walls. Anchored walls, soil nail walls, and anchored soldier pile walls may be suitable for deeper cuts although these walls require either a larger permanent easement or permanent ROW.

Fill walls

Walls constructed in fill locations are typically used for permanent construction and may require large ROW to meet the base width requirements. The necessary fill material may be required to be granular in nature. These walls use bottom up construction and have typical cost effective ranges. Surface conditions must also be considered. For instance, if soft compressible soils are present, walls that can tolerate larger settlements and movements must be considered. MSE walls are generally more economical for fill locations than CIP cantilever walls.

Cut/fill Walls

CIP cantilever and prefabricated modular walls are most suitable in cut/fill situations as the walls are built from bottom up, have narrower base widths and these walls do not rely on soil reinforcement techniques to provide stability. These types of walls are suitable for both cut or fill situations.

14.3.1.3 Site Characteristics

Site characterization should be performed, as appropriate, to provide the necessary information for the design and construction of retaining wall systems. The objective of this characterization is to determine composition and subsurface soil/rock conditions, define engineering properties of foundation material and retained soils, establish groundwater conditions, determine the corrosion potential of the water, and identify any discontinuities or geotechnical issues such as poor bearing capacity, large settlement potential, and/or any other design and construction problems.

Site characterization mainly includes subsurface investigations and analyses. WisDOT's Geotechnical Engineering Unit generally completes the investigation and analyses for all in-house wall design work.

14.3.1.4 Miscellaneous Design Considerations

Other key factors that may influence wall selection include height limitations for specific systems, limit of wall radius on horizontal alignment, and whether the wall is a component of an abutment.

Foundation conditions that may govern the wall selection are bearing capacity, allowable lateral and vertical movements, tolerable settlement and differential movement of retaining wall systems being designed, susceptibility to scour or undermining due to seepage, and long-term maintenance.

14.3.1.5 Right of Way Considerations

Availability of ROW at a site may influence the selection of wall type. When a very narrow ROW is available, a sheet pile wall may be suitable to support an excavation. In other cases, when walls with tiebacks or soil reinforcement are considered, a relatively large ROW may be required to meet wall requirements. Availability of vertical operating space may influence wall selection where piling installation is required and there is not enough room to operate driving equipment.

Section 11-55-5 of the FDM describes the ROW requirement for retaining walls. It requires that all segments of a retaining wall should be under the control of WisDOT. No improvements or utility construction should be allowed in the ROW area of the retaining wall systems.

14.3.1.6 Utilities and Other Conflicts

Feasibility of some wall systems may be influenced by the presence of utilities and buried structures. MSE, soil nailing and anchored walls commonly have conflict with the presence of utilities or buried underground structures. MSE walls should not be used where utilities must stay in the reinforcement zone.

14.3.1.7 Aesthetics

In addition to being functional and economical, the walls should be aesthetically pleasing. Wall aesthetics may influence selection of a particular wall system. However, the aesthetic treatment should complement the retaining wall and not disrupt the functionality or selection of wall type. All permanent walls should be designed with due considerations to the wall aesthetics. Each wall site must be investigated individually for aesthetic needs. Temporary walls should generally be designed with little consideration to aesthetics. Chapter 4 - Aesthetics presents structures aesthetic requirements.

14.3.1.8 Constructability Considerations

Availability of construction materials, site accessibility, equipment availability, form work and temporary shoring, dewatering requirements, labor considerations, complicated alignment changes, scheduling consideration, speed of construction, construction staging/phasing and maintaining traffic during construction are some of the important key factors when evaluating the constructability of each wall system for a specific project site.

In addition, it should also be ensured that the temporary excavation slopes used for wall construction are stable as per site conditions and meet all safety requirements laid by Occupation and Safety Health Administration (OSHA).

14.3.1.9 Environmental Considerations

Selection of a retaining wall system is influenced by its potential environmental impact during and after construction. Some of the environmental concerns during construction may include excavation and disposal of contaminated material at the project site, large quantity of water, corrosive nature of soil/water, vibration impacts, noise abatement and pile driving constraints.

14.3.1.10 Cost

Cost of a retaining wall system is influenced by many factors that must be considered while estimating preliminary costs. The components that influence cost include excavation, structure, procurement of additional easement or ROW, drainage, disposal of unsuitable material, traffic maintenance etc. Maintenance cost also affects overall cost of a retaining wall system. The retaining walls that have least structural cost may not be the most economical walls. Wall selection should be based on overall cost. When feasible, MSE Walls and modular block gravity walls generally cost less than other wall types.

14.3.1.11 Mandates by Other Agencies

In certain project locations, other agency mandates may limit the types of wall systems considered.

14.3.1.12 Requests made by the Public

A Public Interest Finding could dictate the wall system to be used on a specific project.

14.3.1.13 Railing

For safety reasons most walls will require a protective railing. The railing will usually be located behind the wall. The roadway designer will generally determine whether a pedestrian or non-pedestrian railing is required and what aesthetic considerations are needed.

14.3.1.14 Traffic barrier

A traffic barrier should be installed if vehicles, bicycles, or pedestrians are likely to be present on top of the wall. The roadway designer generally determines the need for a traffic barrier.

14.3.2 Wall Selection Guide Charts

Table 14.3-1 and Table 14.3-2 summarize the characteristics for the various wall types that are normally considered during the wall selection process. The tables also present some of the advantages, disadvantages, cost effective height range and other key selection factors. A wall designer can use these tables and the general wall selection criteria discussed in 14.3.1 as a guide. Designers are encouraged to contact the Structures Design Section if they have any questions relating to wall selection for their project.

Wall Type	Temp.	Perm.	Cost Effective Height (ft)	Req'd. ROW	Advantages	Disadvantages
CIP Concrete Gravity		√	3 - 10	0.5H - 0.7H	• Durable • Meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time
CIP Concrete Cantilever		√	6 - 28	0.4H - 0.7H	• Durable meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time & deeper embedment
Reinforced CIP Counterfort		√	26 - 40	0.4H - 0.7H	• Durable • Meets aesthetic requirement • Requires small back fill quantity	• High cost • May need deep foundation • Longer const. time & deeper embedment
Modular Block Gravity		√	3 - 8	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Height limitations
Metal Bin		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or special equipment	• Difficult to make height adjustment in the field
Concrete Crib		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Difficult to make height adjustment in the field
Gabion		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Need large stone quantities • Significant labor
MSE Wall (precast concrete panel with steel reinforcement)		√	10 – 30*	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (modular block and geo-synthetic reinforcement)		√	6 – 22*	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (geotextile/geogrid/ welded wire facing)	√	√	6 – 35*	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill

*WisDOT maximum wall height

Table 14.3-1
Wall Selection Chart for Gravity Walls

Wall Type	Temp.	Perm.	Cost Effective Height (ft)	Req'd. ROW	Water Tightness	Advantages	Disadvantages
Sheet Pile	√	√	6 - 15	Minimal	Fair	• Rapid construction • Readily available	• Deep foundation may be needed • Longer construction time
Soldier Pile	√	√	6 - 28	0.2H - 0.5H	Poor	• Easy construction • Readily available	• High cost • Deep foundation may be needed • Longer construction time
Tangent Pile		√	20 - 60	0.4H - 0.7H	Fair/Poor	• Adaptable to irregular layout • Can control wall stiffness	• High cost • Deep foundation may be needed • Longer construction
Secant Pile		√	14 - 60	0.4H - 0.7H	Fair	• Adaptable to irregular layout • Can control wall stiffness	• Difficult to make height adjustment in the field • High cost
Anchored	√	√	15 - 35	0.4H - 0.7H	Fair/Poor	• Rapid construction	• Difficult to make height adjustment in the field
Soil Nail	√	√	6 - 20	0.4H - 0.7H	Fair	• Option for top-down	• Cannot be used in all soil types • Cannot be used below water table • Significant labor

Table 14.3-2
Wall Selection Chart for Non-Gravity Walls

14.4 General Design Concepts

This section covers the general design standards and criteria to be used for the design of temporary and permanent gravity and non-gravity walls including proprietary and non-proprietary wall systems.

The design criteria for tiered walls that retain other walls or hybrid walls systems requiring special design are not covered specifically in this section.

14.4.1 General Design Steps

The design of wall systems should follow a systematic process applicable for all wall systems and summarized below:

1. Basic Project Requirement: This includes determination of wall alignment, wall geometry, wall function, aesthetic, and project constraints (e.g. right of way, easement during construction, environment, utilities, etc.) as part of the wall development process described in 14.1.

2. Wall Selection: Select wall type based on step 1 and the wall section criteria discussed in 14.3.

3. Geotechnical Investigation: Subsurface investigation and analyses should be performed in accordance with 14.4.4 and Chapter 10 - Geotechnical Investigation to develop foundation and fill material design strength parameters and foundation bearing capacity. Note: this work generally requires preliminary checks performed in step 7, based on steps 4 thru 6.

4. Wall Loading: Determine all applicable loads likely to act on the wall as discussed in 14.4.5.3.

5. Initial Wall Sizing: This step requires initial sizing of various wall components and establishing wall batter which is wall specific and described under each specific wall designs discussed in 14.5 thru 14.13.

6. Wall Design Requirements: Design wall systems using design standards and service life criteria and the *AASHTO Load and Resistance Factor Design* (*AASHTO LRFD*) requirements discussed in 14.4.1 and 14.4.2.

7. Perform external stability, overall stability, and wall movement checks discussed in 14.4.7. These checks will be wall specific and generally performed by the Geotechnical Engineer of record. The stability checks should be performed using the performance limits, load combinations, and the load/resistance factors per *AASHTO LRFD* requirements described in 14.4.5.5 and 14.4.5.6 respectively.

8. Perform internal stability and structural design of the individual wall components and miscellaneous components. These computations are performed by the Designer for non-proprietary walls. For proprietary walls, internal stability is the responsibility of the contractor/supplier after letting.

9. Repeat design steps 4 thru 8 if the required checks are not met.

14.4.2 Design Standards

Retaining wall systems shall be designed in conformance with the current *AASHTO Load and Resistance Factor Design Specifications* (AASHTO LRFD) and in accordance with the WisDOT Bridge Manual. Walls shall be designed to address all limit states.

Wall systems including rock walls and soil nail systems which are not specifically covered by the *AASHTO LRFD* specifications shall be designed using the hierarchy of guidelines presented in this chapter, Allowable Stress Design (ASD) or *AASHTO Load Factor Design* (LFD) methods or the design procedures developed based on standard engineering and/or industry practices. The guidelines presented in this chapter will prevail where interpretation differs. WisDOT's decision shall be final in those cases. The new specifications for the wall designs were implemented October 1st, 2010.

14.4.3 Design Life

All permanent retaining walls and components shall be designed for a minimum service life of 75 years. All temporary walls shall be designed for a period of 36 months or for the project specific duration, whichever is greater. The design of temporary wall systems is the responsibility of the contractor. The temporary walls shall meet all the safety requirements as that of a permanent wall except for corrosion and aesthetics.

14.4.4 Subsurface Exploration

Geotechnical exploration may be needed to explore the soil/rock properties for foundation, retained fill, and backfill soils for all retaining walls regardless of wall height. It is the designer's responsibility to ensure that pertinent soils information, loading conditions, foundation considerations, consolidation potential, settlement and external stability is provided for the wall design.

Before planning a subsurface investigation, it is recommended that any other available subsurface information such as geological or other maps or data available from previous subsurface investigations be studied. Subsurface investigation and analyses should be performed where necessary, in accordance with Chapter 10 - Geotechnical Investigation.

The investigations and analyses may be required to determine or establish the following:

- Nominal bearing pressure, consolidation properties, unit weight and shear strength (drained or undrained strength for fine grained soils) for foundation soils/rocks.
- Shear strength, and unit weight of selected backfill.
- Shear strength and unit weight of random fill or in-situ soil behind selected backfill or wall
- Location of water table

14.4.5 Load and Resistance Factor Design Requirements

14.4.5.1 General

In the LRFD process, wall stability is checked as part of the design process for anticipated failure modes for various types of walls at specified limit states, and the wall components are sized accordingly.

To evaluate the limit states, all applicable design loads are computed as nominal or un-factored loads, than factored using a load factor and grouped to consider the force effect of all loads and load combinations in accordance with **LRFD [3.4.1]**. The factored loads are compared with the factored resistance as part of the stability check in accordance with **LRFD [11.5]** such that the factored resistance is not less than factored loads as presented in **LRFD [1.3.2.1]**

$Q = \sum \eta_i \gamma_I Q_i \leq \phi R_n = R_r$ **LRFD [1.3.2.1-1]**

Where:

η_I = Load modifier (a function of η_D, η_R, assumed 1.0 for retaining walls)

γ_I = Load factor

Q_i = Force effect

Q = Total factored force effect

ϕ = Resistance factor

R_n = Nominal resistance

R_r = Factored resistance = ϕR_n

14.4.5.2 Limit States

The limit states (as defined in **LRFD [3.4.1]**) that must be evaluated as part of the wall design requirements mainly include (1) Strength limit states; (2) Service limit states; and (3) Extreme Event limit states. The fatigue limit state is not used for retaining walls.

Strength limit state is applied to ensure that walls have adequate strength to resist external stability failure due to sliding, bearing resistance failure, etc. and internal stability failure such as pullout of reinforcement, etc. Evaluation of Strength limit states is accomplished by grouping factored loads and comparing to the reduced or factored soil strengths using resistance factors discussed in 14.4.5.6.

Service limit state is evaluated for overall stability and total or differential settlement checks. Evaluation of the Service limit states is usually performed by using expected service loads assuming a factor of 1.0 for nominal loads, a resistance factor of 1.0 for nominal strengths and elastic analyses.

Extreme Event II limit state is evaluated to design walls for vehicular collision forces. In particular, MSE walls having a traffic barrier at the top are vulnerable to damage due to vehicle collision forces and this case for MSE Walls is discussed further in 14.6.3.10.

14.4.5.3 Design Loads

Retaining walls shall be designed to withstand all applicable loads generally categorized as permanent and transient loads.

Permanent loads include dead load DC due to weight of the structural components and non structural components of the wall, dead load DW loads due to wearing surfaces and utilities, vertical earth pressure EV due to dead load of earth, horizontal earth pressure EH and earth surcharge loads ES. Applied earth pressure and earth pressure surcharge loads are further discussed in 14.4.5.4.

The transient loads include, but are not limited to, water pressure WA, live load surcharge LS, and forces caused by the deformations due to shrinkage SH, creep CR and settlement caused by the foundation SE.

These loads should be computed in accordance with **LRFD [3.4]** and **LRFD [11.0].** Only loads applicable for each specific wall type should be considered in the engineering analyses.

14.4.5.4 Earth Pressure

Determination of earth pressure will depend upon types of wall structure (gravity, semi gravity, reinforced earth wall, cantilever or anchored walls, etc.), wall movement, wall geometry, wall friction, configuration, retained soil type, ground water conditions, earth surcharge, and traffic and construction related live load surcharge. In general, earth pressure on retaining walls shall be calculated in accordance with **LRFD [3.11.5]**. Earth pressure that will develop on walls includes active, passive or at-rest earth pressure.

<u>Active Earth Pressure</u>

The active earth pressure condition exists when a retaining wall is free to rotate away from the retained backfill. There are two earth pressure theories available for determining the active earth pressure coefficient (K_a); Rankine and Coulomb earth pressure theories. A detailed discussion of Rankine and Coulomb theories can be found in *Foundation Design- Principles and Practices*; by Donald P. Cudoto or *Foundation Analysis and Design*, 5th Edition by Joseph E. Bowles as well as other standard text books on this subject.

Rankine earth pressure makes assumptions that the retained soil has a horizontal surface, the failure surface is a plane and that the wall is smooth (i.e. no friction). Rankine earth pressure theory is the preferred method for developing the active earth pressure coefficient; however, where wall friction is an important consideration or where sloping surcharge loads are considered, Coulomb earth pressure theory may be used. The use of Rankine theory will cause a slight over estimation of $K_{a,}$ therefore, increasing the pressure on the wall resulting in a more conservative design.

Walls that are cast-in-place (CIP) semi gravity concrete cantilever referred, hereafter, as CIP cantilever, Mechanically Stabilized Earth (MSE), modular block gravity, soil nailing, soldier-pile and sheet-pile walls are typically considered flexible enough to justify using an active earth pressure coefficient.

At-Rest Earth Pressure

In the at-rest earth pressure (K_o) condition, the top of the wall is not allowed to deflect or rotate; therefore, requiring the wall to support the full pressure of the soil behind the wall.

The at-rest earth pressure coefficient shall be used to calculate the lateral earth pressure for non-yielding retaining walls restrained from rotation and/or lateral translation in accordance with **LRFD [3.11.5.2]**. Non-yielding walls include integral abutment walls, or retaining walls resting on bedrock or pile foundation.

Passive Earth Pressure

The development of passive earth pressure (K_p) requires a retaining wall to move into or toward the soil. As with the active earth pressure, Rankine earth pressure is the preferred method to be used to develop passive earth pressure coefficient. The use of Rankine theory will cause an under estimation of K_p, therefore resulting in a more conservative design. Coulomb earth pressure theory may be used if the appropriate conditions exist at a site; however, the designer is required to understand the limitations on the use of Coulomb earth pressure theory as applied to passive earth pressures.

Neglect any contribution from passive earth pressure in stability calculations unless the base of the wall extends below the depth to which foundation soil or rock could be weakened or removed by freeze-thaw, shrink-swell, scour, erosion, construction excavation, or any other means. In wall stability calculations, only the embedment below this depth, known as the effective embedment depth, shall be considered when calculating the passive earth pressure resistance. This is in accordance with **LRFD [11.6.3.5].**

14.4.5.4.1 Earth Load Surcharge

The effect of earth load surcharge including uniform, strip, and point loads shall be computed in accordance with **LRFD [3.11.6.1]** and **LRFD [3.11.6.2].**

14.4.5.4.2 Live Load Surcharge

Increased earth pressure on a wall occurs due to vehicular loading on top of the retained earth including operation of large or heavily-loaded cranes, staged equipment, soil stockpile or material storage, or any surcharge loads behind the walls. Earth pressure from live load surcharge shall be applied when a vehicular load is within one half of the wall height behind the back face of the wall or reinforced soil mass for MSE walls, in accordance with **LRFD [3.11.6.4].** In most cases, surcharge load can be modeled by assuming 2 ft of fill.

WisDOT policy item:

The equivalent height of soils for vehicular loading on retaining walls parallel to the traffic shall be 2.0 feet, regardless of the wall height. For standard unit weight of soil equal to 120 pcf, the resulting live load surcharge is 240 psf. Walls without traffic shall be designed for a live load surcharge of 100 psf to account for construction live loads.

14.4.5.4.3 Compaction Loads

Pressure induced by the compaction load can extend to variable depths due to the total static and dynamic forces exerted by compaction equipment. The effect of increased lateral earth pressure due to compaction loads during construction should be considered when compaction equipment is operated behind the wall. The compaction load surcharge effect is minimized by WISDOT standard specifications that require small walk behind compactors within 3 ft of the wall.

14.4.5.4.4 Wall Slopes

The slopes above and below the wall can significantly affect the earth pressures and wall stability. Slopes above the wall will influence the active earth pressure; slopes at the toe of the wall influences the passive earth pressures. In general, the back slope behind the wall should be no steeper than 2:1 (H:V). Where possible, a 4.0 ft wide horizontal bench should be provided at the front face of the wall.

14.4.5.4.5 Loading and Earth Pressure Diagrams

Loading and earth pressure diagrams are developed to compute nominal (unfactored) loads and moments. All applicable loads described in 14.4.5.3 and 14.4.5 shall be considered for computing nominal loads. For a typical wall, the force diagram for the earth pressure should be developed using a triangular distribution plus additional pressures resulting from earth or live load surcharge, water pressure, compaction etc. as discussed in 14.4.5.4.

The engineering properties for selected fill, concrete and steel are given in 14.4.6. The foundation and retained earth properties are selected as per discussions in 14.4.4 . One of the three cases is generally applicable for the development of loading diagrams and earth pressures:

1. Horizontal backslope with traffic surcharge
2. Sloping backslope
3. Broken backslope

Loading diagrams for CIP cantilever, MSE, modular block gravity, and prefabricated modular walls are shown for illustration. The designer shall develop loading diagrams as applicable.

CIP cantilever wall with sloping surcharge

For CIP cantilever walls, lateral active earth pressure shall be computed using Coulomb's theory for short heels or using Rankine theory for very long heels in accordance with the criteria presented in **LRFD [3.11.5.3]** and **LRFD [C3.11.5.3]**.

Walls resting on rock or batter piles can be designed for active earth pressure, based on WisDOT policy and in accordance with **LRFD [3.11.5.2].** Effect of the passive earth pressure on the front face of the wall shall be neglected in stability computation, unless the base of the wall extends below depth of maximum scour, freeze thaw or other disturbances in accordance with **LRFD [11.6.3.5]**.

Effect of surcharge loads ES present at the surface of the backfill of the wall shall be included in the analysis in accordance with 14.4.5.4.1. Walls with horizontal backfill shall be designed for live load surcharge in accordance with 14.4.5.4.2.

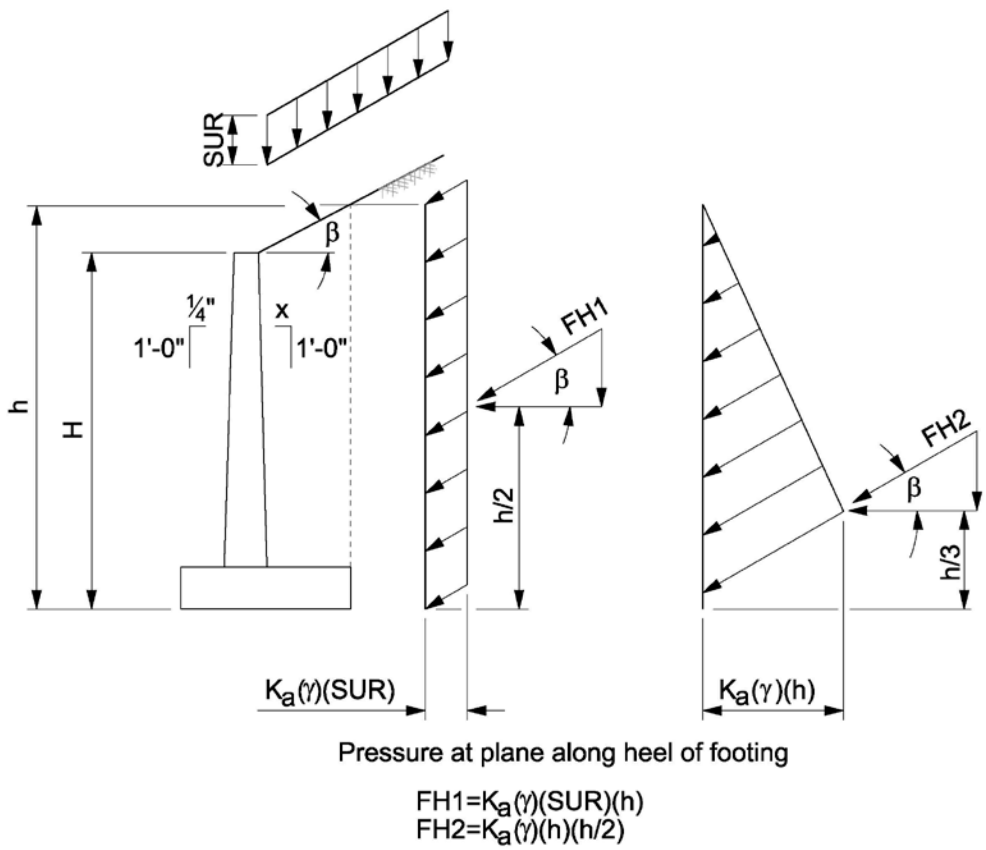


Figure14.4-1
Loading Diagram for a Cantilever Retaining Wall with Surcharge Loading

MSE Walls

The loading and earth pressure diagram for an MSE wall shall be developed in accordance with **LRFD [11.11.2]** and described below for the three conditions defined earlier in this section.

MSE Wall with Horizontal Backslope and Traffic Surcharge

Figure 14.4-2 shows a procedure to estimate the earth pressure. The active earth pressure for horizontal backslope is computed using a simplified version of Coulomb theory

$$K_a = \tan^2 (45 - \phi_f / 2)$$

Where:

K_a = Coefficient of active earth pressure

φ_f = Angle of internal friction of retained earth

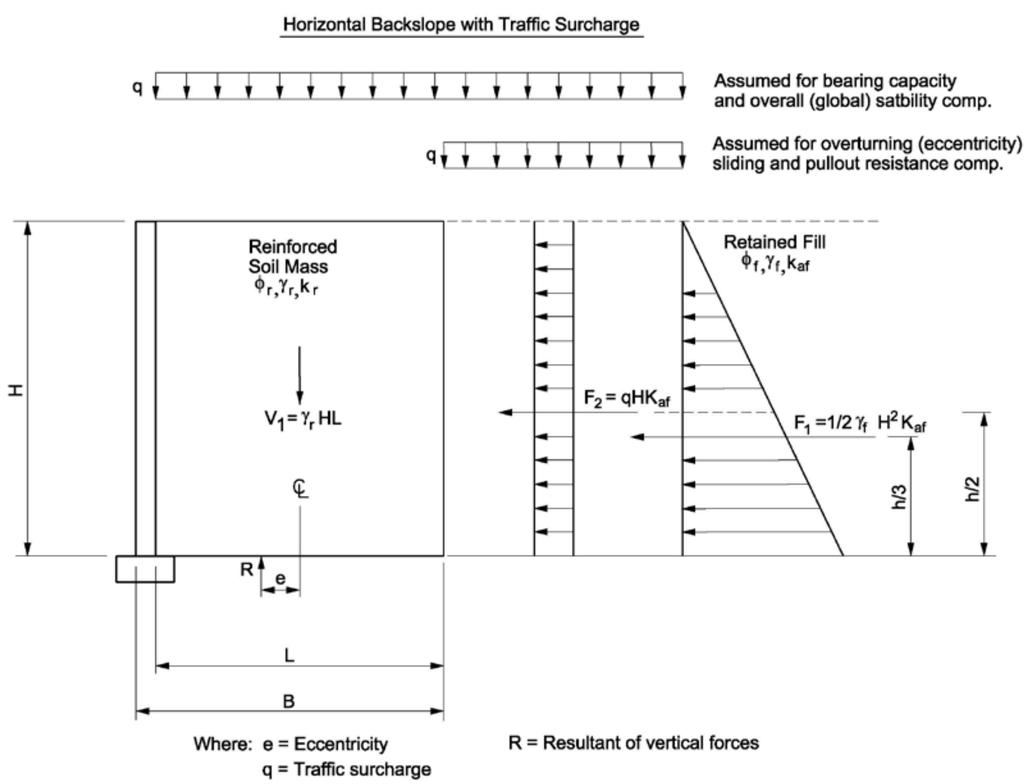


Figure 14.4-2
MSE Walls Earth Pressure for Horizontal Backslope with Traffic Surcharge
(Source *AASHTO LRFD*)

MSE Wall with Sloping Surcharge

The active earth pressure coefficient K_a is computed using Coulomb's equation. The force on the rear of the reinforced soil mass (F_t) and the resulting horizontal (F_h) and vertical (F_v) forces are determined from the following equations:

$$F_T = 1/2\ \gamma_f h^2 K_{af}$$

$$F_h = F_t \cos \beta$$

$$F_v = F_t \sin \beta$$

Where:

γ_f	=	Unit weight of retained fill material
β	=	Slope angle of backfill behind wall
δ	=	Angle of friction between retained backfill and reinforced backfill
h	=	See Figure 14.4-3
K_{af}	=	Use Coulomb's equation

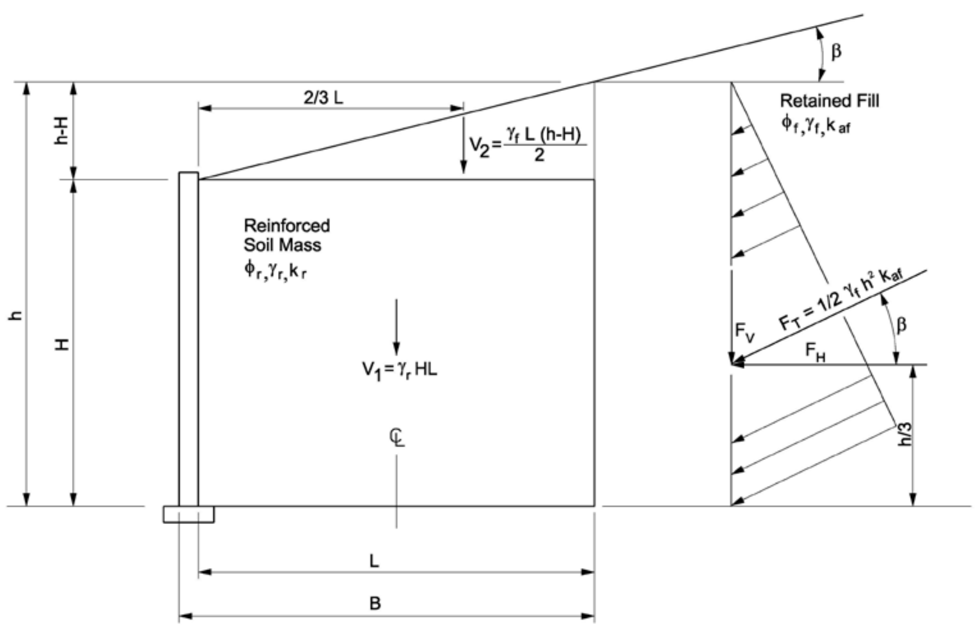


Figure 14.4-3
MSE Walls Earth Pressure for Sloping Backfill
(Source *AASHTO LRFD*)

MSE Wall with Broken Backslope

For broken backslopes, the active earth pressure coefficient is determined using Coulomb's equation except that surcharge angle β and interface angle δ is substituted with infinite slope angle I. Force, F_t, is determined using:

$F_t = 1/2\ \gamma_f h^2 K_{af}$

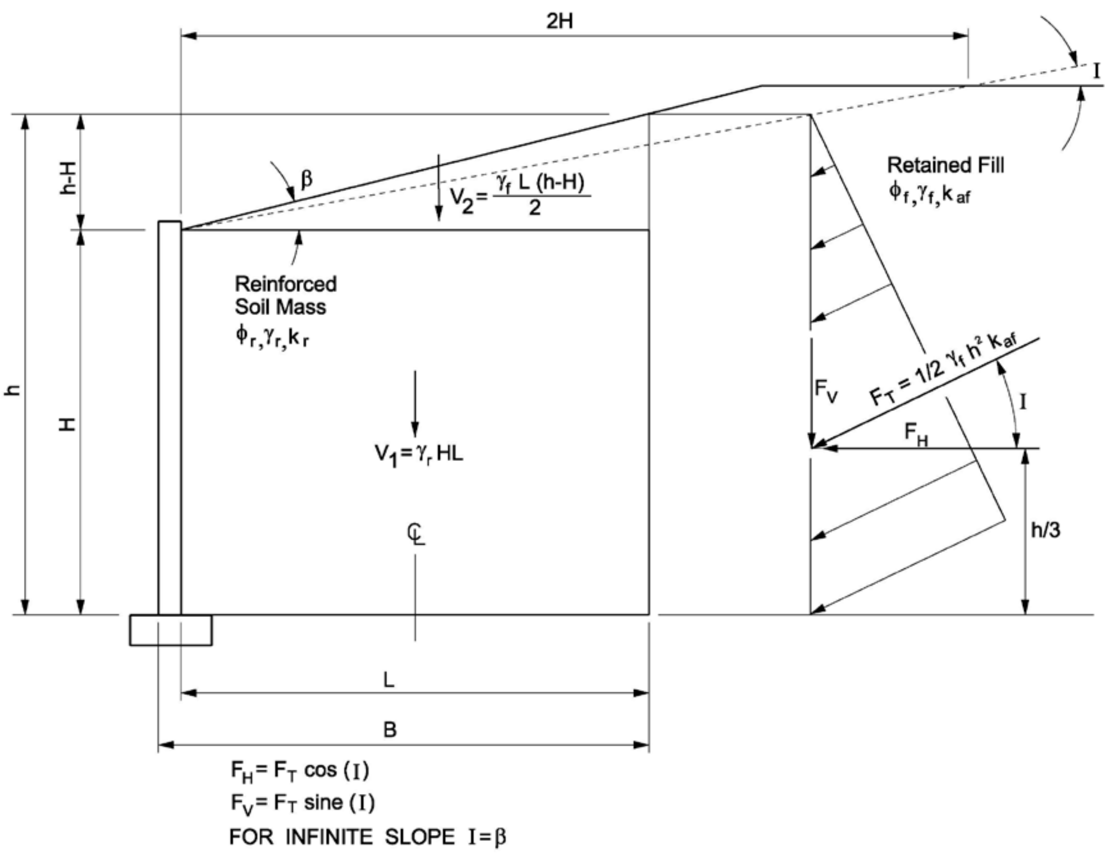


Figure 14.4-4
MSE Walls Earth Pressure for Broken Backfill
(Source *AASHTO LRFD*)

Modular Block Gravity Wall with Sloping Surcharge

When designing a "Modular Block Gravity Wall" without setback and with level backfill, the active earth pressure coefficient may be determined using Rankine theory from the following formula.

$K_a = \tan^2(45 - \phi_f / 2)$

When designing a "Modular Block Gravity Wall" with setback, the active earth pressure coefficient K_a shall be determined from the following Coulomb formula. The interface friction angle between the blocks and soil behind the blocks is assumed to be zero.

$$K_a = \frac{\cos^2(\phi_f + A)}{\cos^2 A \cos A (1 + (Z/Y)^{1/2})^2}$$

Where:

Z = $\sin \phi_f \sin(\phi_r - \beta)$

Y = $\cos A \cos(A+\beta)$

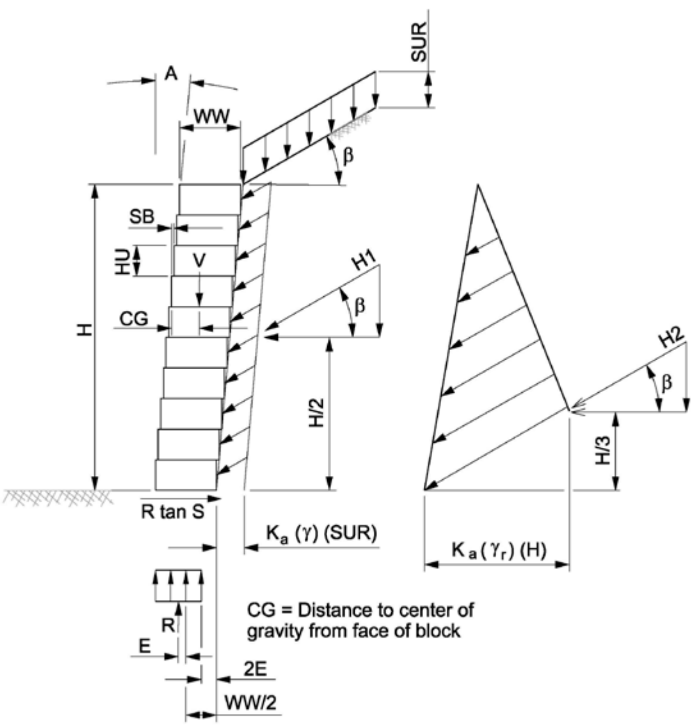


Figure 14.4-5
Modular Block Gravity Wall Analysis

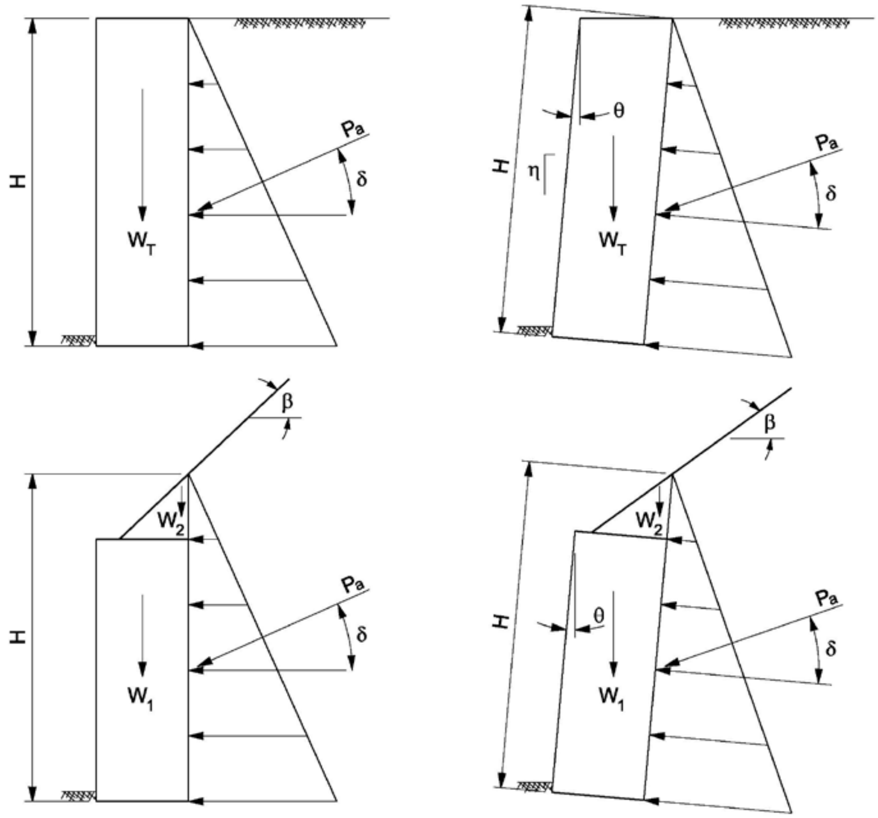
No live load traffic and live load surcharge shall be allowed on modular block gravity walls although they are designed for a minimum live load of 100psf. The density of the blocks is assumed to be 135 pcf and the drainage aggregate inside or between the blocks 120 pcf. The forces acting on a modular block gravity wall are shown in Figure 14.4-5.

Prefabricated Modular Walls

Active earth pressure shall be determined by multiplying vertical loads by the coefficient of active earth pressure (K_a) and using Coulomb earth pressure theory in accordance with **LRFD [3.11.5.3] and LRFD [3.11.5.9]**. See Figure 14.4-6 for earth pressure diagram.

When the rear of the modules form an irregular surface (stepped surface), pressures shall be computed on an average plane surface drawn from the lower back heel of the lowest module as shown in Figure 14.4-7

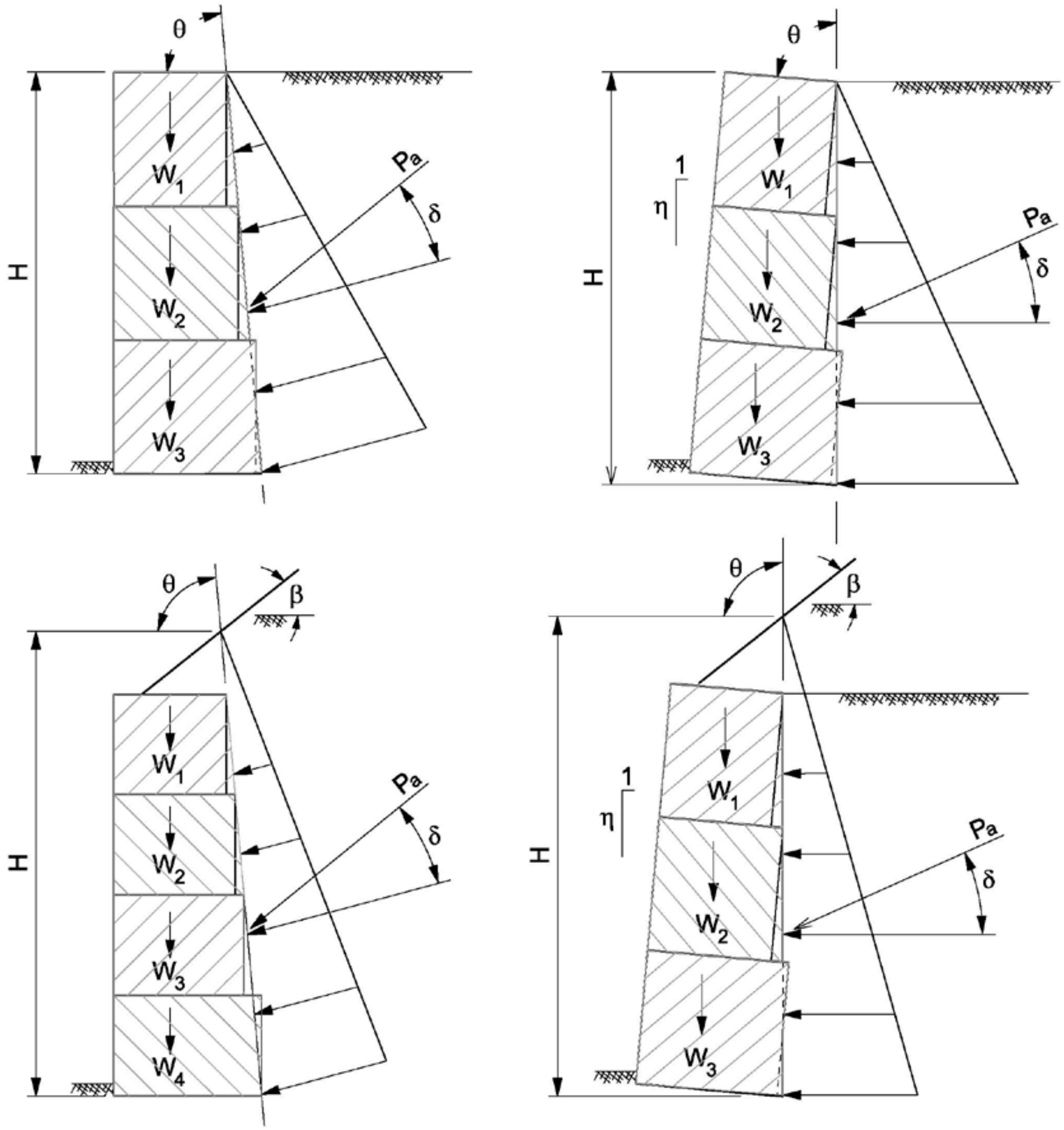
Effect of the backslope soil surcharge and any other surcharge load imposed by existing structure should be accounted as discussed in 14.4.5.4. Trial wedge or Culmann method may also be used to compute the lateral earth pressure as presented in the *Foundation Analysis and Design*, 5th Edition (J. Bowles, 1996).



P_a @ H/3
$P_a = \frac{1}{2} \gamma H^2 K_a$

$$K_a = \frac{\cos^2(\phi+\theta)}{\cos^2\theta \cos(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\theta-\delta)\cos(\theta+\beta)}}\right]^2}$$

Figure 14.4-6
Lateral Earth Pressure on Concrete Modular Systems of Constant Width
(Source *AASHTO LRFD*)



$$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2}$$

Figure 14.4-7
Lateral Earth Pressure on Concrete Modular Systems of Variable Width
(Source *AASHTO LRFD*)

14.4.5.5 Load factors and Load Combinations

The nominal loads and moments as described in 14.4.5.4.5 are factored using load factors found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The load factors applicable for most wall types considered in this chapter are given in Table 14.4-1. Load factors are selected to produce a total extreme factored force effect, and for each loading combination, both maximum and minimum extremes are investigated as part of the stability check, depending upon the expected wall failure mechanism.

Direction of Load	Load Type	Load Factor, γ_i		
		Strength I Limit		Service I Limit
		Maximum	Minimum	
Load Factors for Vertical Loads	Dead Load of Structural Components and Non-structural attachments DC	1.25	0.90	1.00
	Earth Surcharge Load ES	1.50	0.75	1.00
	Vertical Earth Load EV	1.35	1.00	1.00
	Water Load WA	1.00	1.00	1.00
	Live Load Surcharge LS	1.75	0.0	1.00
	Dead Load of Wearing Surfaces and Utilities DW	1.50	0.65	1.00
Load Factors for Horizontal Loads	Horizontal Earth Pressure EH			
	Active	1.50	0.90	1.00
	At-Rest	1.35	0.90	1.00
	Passive	1.35	NA	1.00
	Earth Surcharge ES	1.50	0.75	1.00
	Live Load Surcharge LS	1.75	1.75	1.00

Table 14.4-1
Load Factors

The factored loads are grouped to consider the force effect of all loads and load combinations for the specified load limit state in accordance with **LRFD [3.4.1].** Figure 14.4-8 illustrates the load factors and load combinations applicable for checking sliding stability and eccentricity for a cantilever wall at the Strength I limit state. This figure shows that structure weight DC is factored by using a load factor of 0.9 and the vertical earth load EV is factored by using a factor of 1.0. This causes contributing stabilizing forces against sliding to have a minimum force effect. At the same time, the horizontal earth load is factored by 1.5 resulting in maximum force effect for computing sliding at the base.

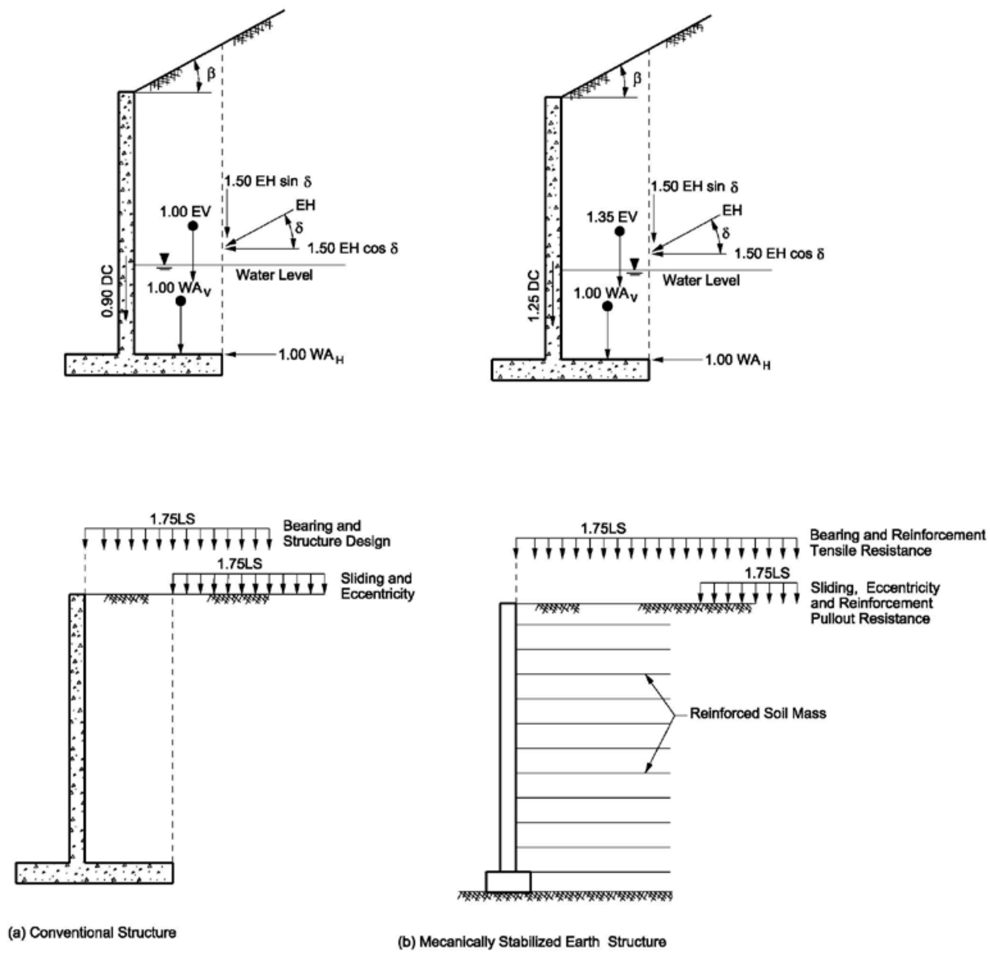


Figure 14.4-8
Application of Load Factors
(Source *AASHTO LRFD*)

14.4.5.6 Resistance Requirements and Resistance Factors

The wall components shall be proportioned by the appropriate methods so that the factored resistance as shown in **LRFD [1.3.2.1-1]** is no less than the factored loads, and satisfy criteria in accordance with **LRFD [11.5.4]** and **LRFD [11.6] thru [11.11]**. The factored resistance R_r is computed as follows: $R_r = \phi R_n$

Where

R_r	=	Factored resistance
R_n	=	Nominal resistance recommended in the Geotechnical Report
ϕ	=	Resistance factor

The resistance factors shall be selected in accordance with **LRFD [Tables 10.5.5.2.2-1, 10.5.5.2.3-1, 10.5.5.2.4-1, 11.5.6.1]**. Commonly used resistance factors for retaining walls are presented in Table 14.4-2.

14.4.6 Material Properties

The unit weight and strength properties of retained earth and foundation soil/rock (γ_f) are supplied in the geotechnical report and should be used for design purposes. Unless otherwise noted or recommended by the Designer or Geotechnical Engineer of record, the following material properties shall be assumed for the design and analysis if the selected backfill, concrete, and steel conforms to the WisDOT's *Standard Construction Specifications*:

Granular Backfill Soil Properties:

Internal Friction angle of backfill ϕ_f = 30 degrees

Backfill cohesion c = 0 psf

Unit Weight γ_f = 120 pcf

Concrete:

Compressive strength, f'_c at 28 days = 3500 psi

Unit Weight = 150 pcf

Steel reinforcement:

Yield strength f_y = 60,000 psi

Modulus of elasticity E_s = 29,000 ksi

Wall-Type and Condition		Resistance Factors
Mechanically Stabilized Earth Walls, Gravity Walls, and Semi-Gravity		
Bearing resistance	• Gravity & Semi-gravity • MSE	0.55 0.65
Sliding		1.00
Tensile resistance of metallic reinforcement and connectors	Strip reinforcement • Static loading Grid reinforcement • Static loading	0.75 0.65
Tensile resistance of geo-synthetic reinforcements and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
Prefabricated Modular Walls		
Bearing		**LRFD [10.5]**
Sliding		**LRFD [10.5]**
Passive resistance		**LRFD [10.5]**
Non-Gravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		**LRFD [10.5]**
Passive resistance of vertical elements		0.75
Pullout resistance of anchors	• Cohesionless soils • Cohesive soils • Rock	0.65 0.70 0.50
Pullout resistance of anchors	• Where proof tests are conducted	1.00
Tensile resistance of anchor tendons	• Mild steel • High strength steel	0.90 0.80
Flexural capacity of vertical elements		0.90

Table 14.4-2
Resistance Factors
Source **LRFD [Table 11.5.7-1]**

14.4.7 Wall Stability Checks

During the design process, walls shall be checked for anticipated failure mechanisms relating to external stability, internal stability (where applicable), movement and overall stability. In general, external and internal stability of the walls should be investigated at Strength limit states, in accordance with **LRFD [11.5.1]**. In addition, investigate the wall stability for excessive vertical and lateral displacement and overall stability at the Service limit states in accordance with **LRFD [11.5.2]**. Figure 14.4-2 thru Figure 14.4-14 show anticipated failure mechanisms for various types of walls.

14.4.7.1 External Stability

The external stability should be satisfied (generally performed by the Geotechnical Engineer) for all walls. The external stability check should include failure against lateral sliding, overturning (eccentricity), and bearing pressure failure as applicable for gravity or non-gravity wall systems in accordance with **LRFD [11.5.3]**. External stability checks should be performed at the Strength I limit state.

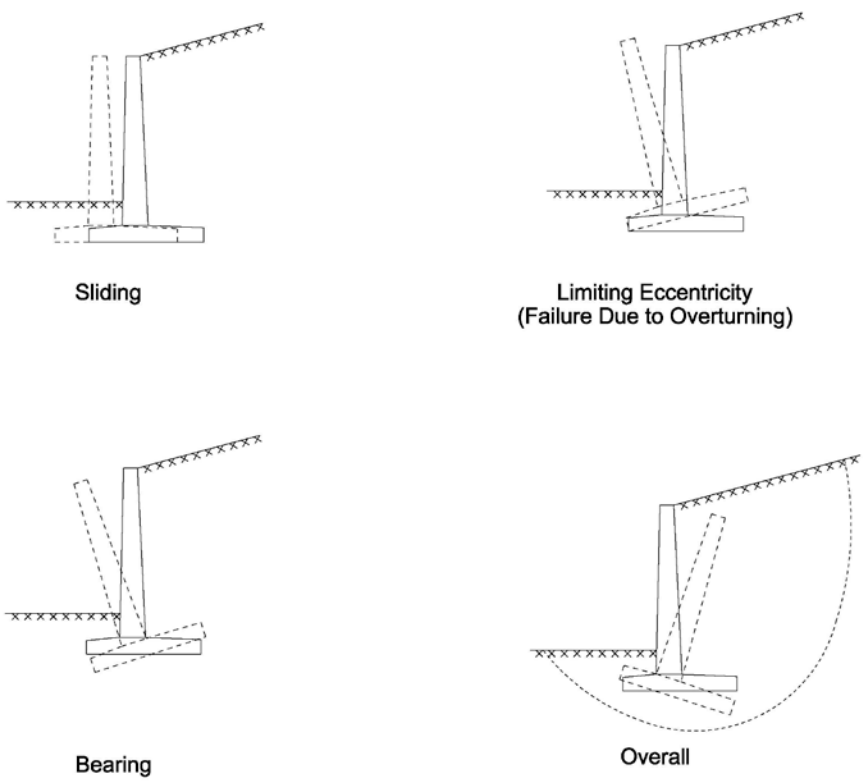


Figure14.4-9
External Stability Failure of CIP Semi-Gravity Walls

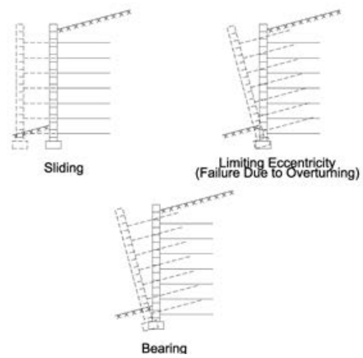


Figure 14.4-10
External Stability Failure of MSE Walls

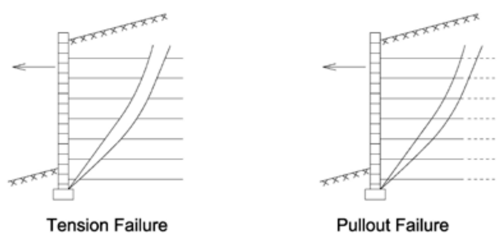


Figure 14.4-11
Internal Stability Failure of MSE Walls

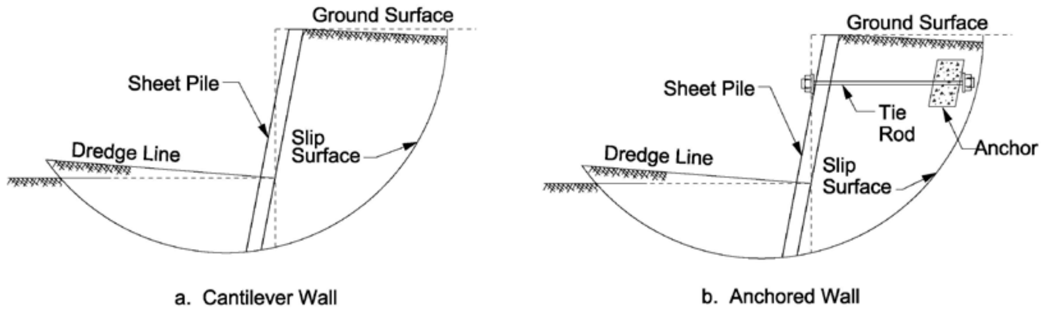


Figure 14.4-12
Deep Seated Failure of Non-Gravity Walls

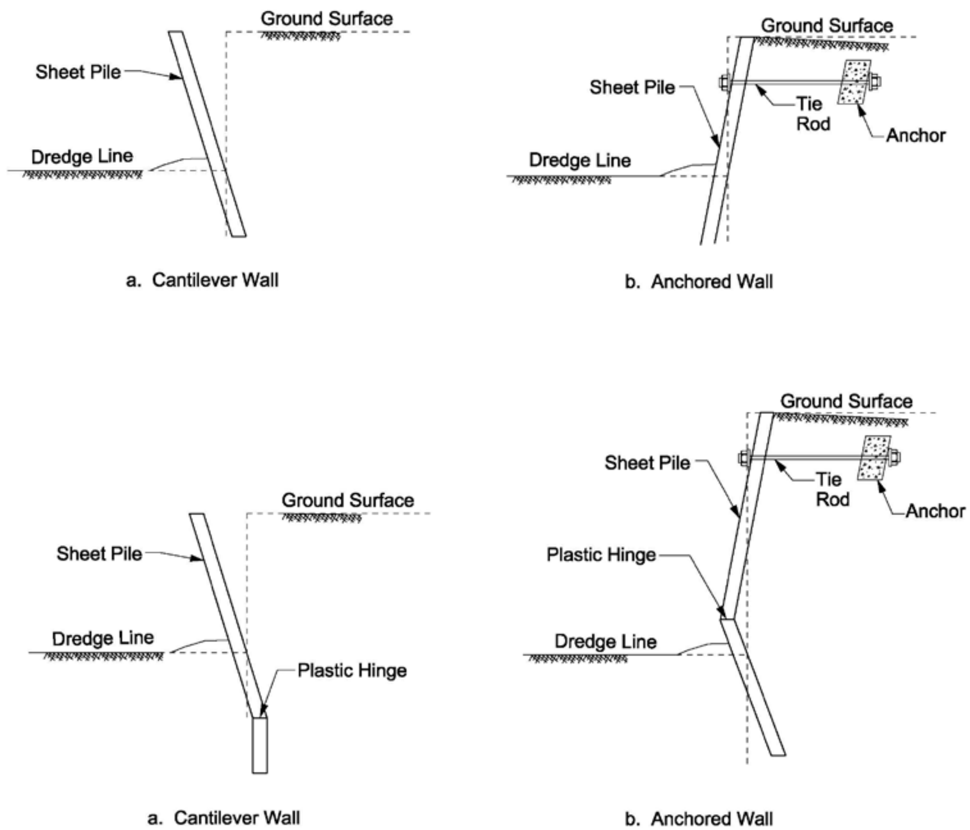


Figure 14.4-13
Flexural Failure of Non-Gravity Walls

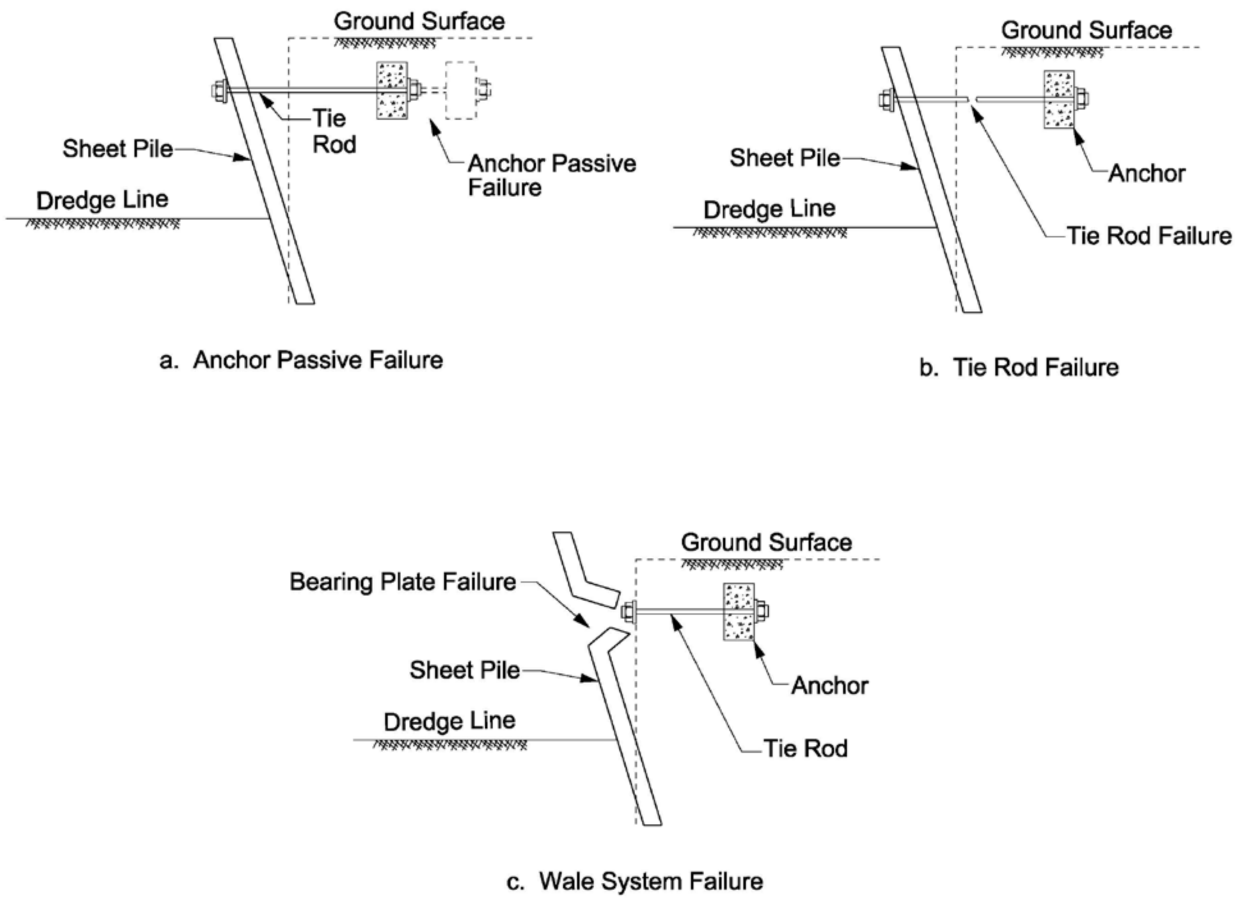


Figure 14.4-14
Flexural Failure of Non-Gravity Walls

14.4.7.2 Wall Settlement

Retaining walls shall be designed for the effects of total and differential foundation settlement at the Service I limit state, in accordance with **LRFD [11.5.2]** and 11.2. Maximum tolerable retaining wall total and differential foundation settlements are controlled largely by the potential for cosmetic and/or structural damage to facing elements, copings, barrier, guardrail, signs, pavements, utilities, structure foundations, and other highway appurtenances supported on or near the retaining wall.

14.4.7.2.1 Settlement Guidelines

The following table provides guidance for maximum tolerable vertical and total differential Settlement for various retaining wall types where Δh is the total settlement in inches and

Wall Type	Total Settlement Δh in inches	Total Differential Settlement Δh1:L (in/in)
CIP semi-gravity cantilever walls	1-2	1:500
MSE walls with large pre-cast panel facing (panel front face area >30ft^2)	1-2	1:500
MSE walls with small pre-cast panel facing (panel front face area <30ft^2)	1-2	1:300
MSE walls with full-height cast-in-panel facing	1-2	1:500
MSE walls with modular block facing	2-4	1:200
MSE walls with geotextile /welded-wire facing	4-8	1:50-1:60
Modular block gravity walls	1-2	1:300
Concrete Crib walls	1-2	1:500
Bin walls	2-4	1:200
Gabion walls	4-6	1:50
Non-gravity cantilever and anchored walls	1-2.5	----

Table 14.4-3
Maximum Tolerable Settlement Guidelines for Retaining Walls

Δh1:L is the ratio of the difference in total vertical settlement between two points along the wall base to the horizontal distance between the two points(L). It should be noted that the tolerance provided in Table 14.4-3 are for guidance purposes only. More stringent tolerances may be required to meet project-specific requirements.

14.4.7.3 Overall Stability

Overall stability of the walls shall be checked at the Service I limit state using appropriate load combinations and resistance factors in accordance with **LRFD [11.6.2.3].** The stability is evaluated using limit state equilibrium methods. The Modified Bishop, Janbu or Spencer method may be used for the analysis. The analyses shall investigate all potential internal, compound and overall shear failure surfaces that penetrate the wall, wall face, bench, back-cut, backfill, and/or foundation zone. The overall stability check is performed by the Geotechnical Engineering Unit for WISDOT designed walls.

14.4.7.4 Internal Stability

Internal stability checks including anchor pullout or soil reinforcement failure and/or structural failure checks are also required as applicable for different wall systems. As an example, see Figure 14.4-11 for internal stability failure of MSE walls. Internal stability checks must be performed at Strength Limits in accordance with **LRFD [11.5.3].**

14.4.7.5 Wall Embedment

The minimum wall footing embedment shall be 1.5 ft below the lowest adjacent grade in front of the wall.

The embedment depth of most wall footings should be established below the depths the foundation soil/rock could be weakened due to the effect of freeze thaw, shrink-swell, scour, scour, erosion, erosion, construction excavation. The potential scour elevation shall be established in accordance with 11.2.2.1.1 of the Bridge Manual.

The final footing embedment depth shall be based on the required geotechnical bearing resistance, wall settlement limitations, and all internal, external, and overall (global) wall stability requirements in *AASHTO LRFD* and the *Bridge Manual*.

14.4.7.6 Wall Subsurface Drainage

Retaining wall drainage is necessary to prevent hydrostatic pressure and frost pressure. Inadequate wall sub-drainage can cause premature deterioration, reduced stability and collapse or failure of a retaining wall.

A properly designed wall sub-drainage system is required to control potentially damaging hydrostatic pressures and seepage forces behind and around a wall. A redundancy in the sub-drainage system is required where subsurface drainage is critical for maintaining retaining wall stability. This is accomplished using a pervious granular fill behind the wall.

Pipe underdrain must be provided to drain this fill. Therefore, “Pipe Underdrain Wrapped 6-Inch” is required behind all gravity retaining walls where seepage should be relieved. Gabion walls do not require a pipe drain system as these are porous due to rock fill. It is best to place the pipe underdrain at the top of the wall footing elevation. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain could be placed higher.

Pipe underdrains and weep holes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks. Consideration should be given to connect the pipe underdrain to the storm sewer system.

14.4.7.7 Scour

The probable depth of scour shall be determined by subsurface exploration and hydraulic studies if the wall is located in flood prone areas. Refer to 11.2.2.1.1 for guidance related to scour vulnerability and design of walls. All walls with shallow foundations shall be founded below the scour elevation.

14.4.7.8 Corrosion

All metallic components of WISDOT retaining wall systems subjected to corrosion, should be designed to last through the designed life of the walls. Corrosion protection should be designed in accordance with the criteria given in **LRFD [11.10.6].** In addition, **LRFD [11.8.7] thru [11.10]** also include design guidance for corrosion protection on non-gravity cantilever walls, anchored walls and MSE walls respectively.

14.4.7.9 Utilities

Walls that have or may have future utilities in the backfill should minimize the use of soil reinforcement. MSE, soil nail, and anchored walls commonly have conflicts with utilities and should not be used when utilities must remain in or below the reinforced soil zone unless there is no other wall option. Utilities that are encapsulated by wall reinforcement may not be accessible for replacement or maintenance. Utility agreements should specifically address future access if wall reinforcing will affect access.

14.4.7.10 Guardrail and Barrier

Guardrail and barrier shall meet the requirements of the Chapter 30 - Railings, *Facilities Development Manual,* Standard Plans, and *AASHTO LRFD*. In no case shall guardrail be placed through MSE wall or reinforced slope soil reinforcement closer than 3 ft from the back of the wall facing elements. Furthermore, the guard rail posts shall be installed through the soil reinforcement in a manner that prevents ripping, damage and distortion of the soil reinforcement. In addition, the soil reinforcement shall be designed to account for the reduced cross-section resulting from the guardrail post holes.

14.5 Cast-In-Place Concrete Cantilever Walls

14.5.1 General

A cast-in-place, reinforced concrete cantilever wall is a semi-gravity wall that consists of a base slab or footing from which a vertical wall or stem extends upward. Reinforcement is provided in both members to supply resistance to bending. These walls are generally founded on good bearing material. Cantilever walls shall not be used without pile support if the foundation stratum is prone to excessive vertical or differential settlement, unless subgrade improvements are made. Cantilever walls are typically designed to a height of 28 feet. For heights exceeding 28 feet, consideration should be given to providing a counterfort. Design of counterfort CIP walls is not covered in this chapter.

CIP cantilever walls shall be designed in accordance with *AASHTO LRFD,* design concepts presented in 14.4 and the *WisDOT Standard Specifications* including the special provisions.

14.5.2 Design Procedure for Cast-in-Place Concrete Cantilever Walls

The CIP wall shall be designed to resist lateral pressure caused by supported earth, surcharge loads and water in accordance with **LRFD [11.6].** The external stability, settlement, and overall stability shall be evaluated at the appropriate load limit states in accordance with **LRFD [11.5.5],** to resist anticipated failure mechanism. The structural components mainly stem and footing should be designed to resist flexural resistance in accordance with **LRFD [11.6.3].**

Figure 14.5-1 shows possible external stability failure and deep seated rotational failure mechanisms of CIP cantilever walls that must be investigated as part of the stability check.

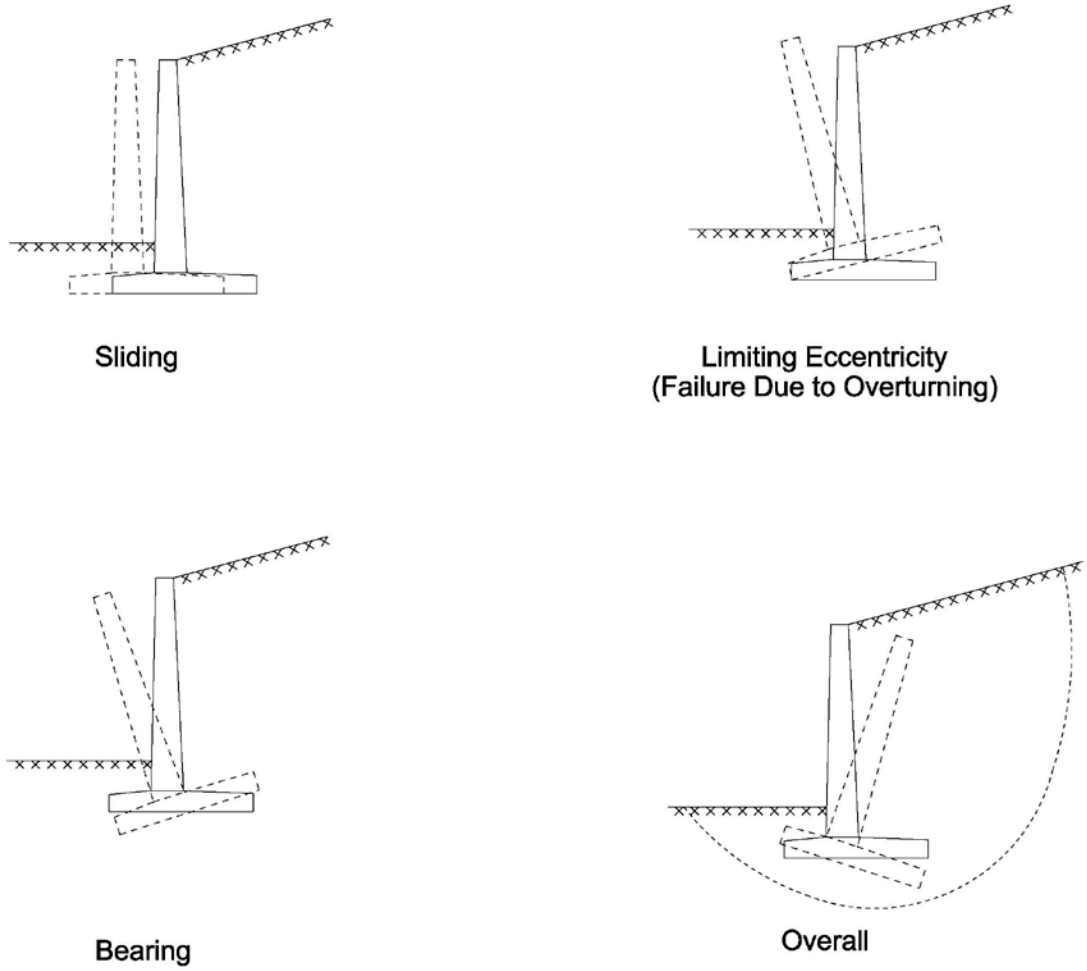


Figure 14.5-1
CIP Semi-Gravity Wall Failure Mechanism

14.5.2.1 Design Steps

The general design steps discussed in 14.4.1 shall be followed for the wall design. These steps as applicable for CIP cantilever walls are summarized below.

1. Establish project requirements including wall height, geometry and wall location as discussed in 14.1 of this chapter.

2. Perform Geotechnical investigation

3. Develop soil strength parameters

4. Determine preliminary sizing for external stability evaluation

5. Determine applicable unfactored or nominal loads

6. Evaluate factored loads for all appropriate limit states

7. Perform stability check to evaluate bearing resistance, eccentricity, and sliding as part of external stability

8. Estimate wall settlement and lateral wall movement to meet guidelines stated in Table 14.4-3.

9. Check overall stability and revise design, if necessary, by repeating steps 4 to 8.

It is assumed that steps 1, 2 and 3 have been performed prior to starting the design process.

14.5.3 Preliminary Sizing

A preliminary design can be performed using the following guideline.

1. The wall height and alignment shall be selected in accordance with the preliminary plan preparation process discussed in 14.1.

2. Preliminary CIP wall design may assume a stem top width of 12 inches. Stem thickness at the bottom is based on load requirements and/or batter. The front batter of the stem should be set at ¼ inch per foot for stem heights up to 28 feet. For stem heights from 16 feet to 26 feet inclusive, the back face batter shall be a minimum of ½ inch per foot, and for stem heights of 28 ft maximum and greater, the back face shall be ¾ inch per foot per stability requirements.

3. Minimum Footing thickness for stem heights equal to or less than 10 ft shall be 1.5 ft and 2.0 ft when the stem height exceeds 10 ft or when piles are used.

4. The base of the footing shall be placed below the frost line, or 4 feet below the finished ground line. Selection of shallow footing or deep foundation shall be based on the geotechnical investigation, which should be performed in accordance with guidelines presented in Chapter 11 - Foundation Support.

5. The final footing embedment shall be based on wall stability requirements including bearing resistance, wall settlement limitations, external stability, internal stability and overall stability requirements.

6. If the finished ground line is on a grade, the bottom of footings may be sloped to a maximum grade of 12 percent. If the grade exceeds 12 percent, place the footings level and use steps.

The designer has the option to vary the values of each wall component discussed in steps 2 to 6 above, depending on site requirements and to achieve economy. See Figure 14.5-2 for initial wall sizing guidance.

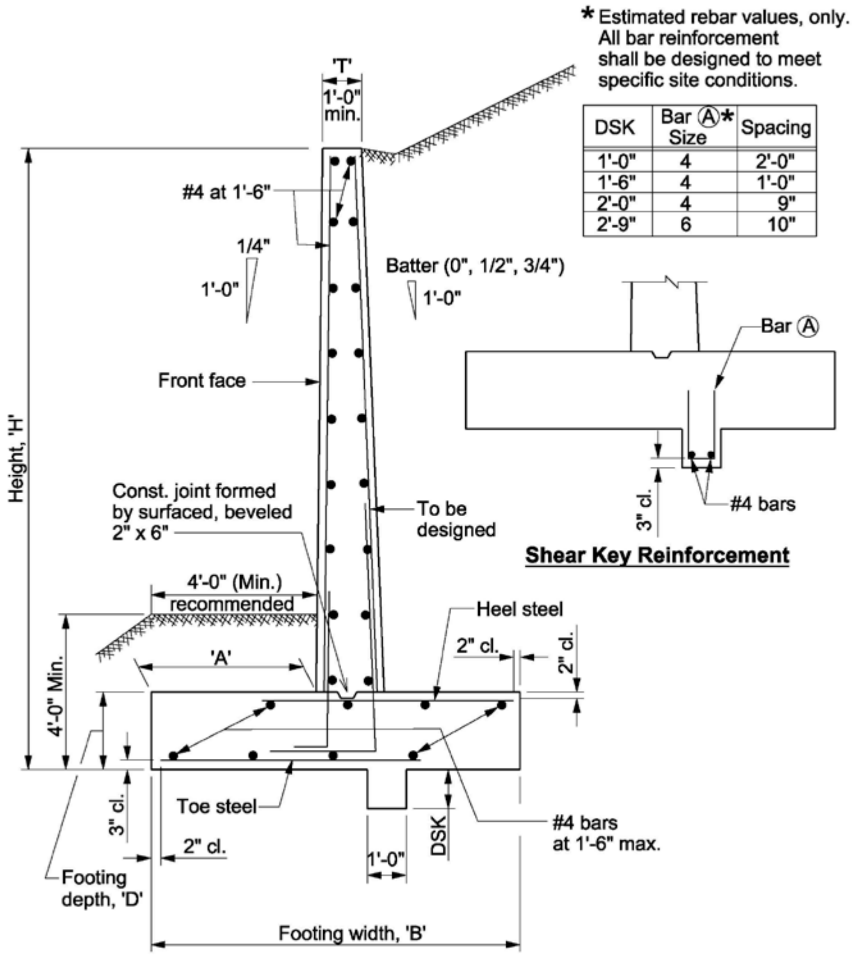


Figure 14.5-2
CIP Walls General Details

14.5.3.1 Wall Back and Front Slopes

CIP walls shall not be designed for backfill slope steeper than 2:1(H:V). Where practical, walls shall have a horizontal bench of 4.0 feet wide at the front face.

14.5.4 Unfactored and Factored Loads

Unfactored loads and moments are computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure14.4-1 for the earth pressure is developed assuming a triangular distribution plus additional pressures resulting from earth surcharge, water pressure, compaction or any other loads, etc. The material properties for backfill soil, concrete and steel are given in 14.4.6. The foundation and retained earth properties as recommended in the Geotechnical Report shall be used for computing nominal loads.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. Figure 14.4-8 shows load factor and load combinations along with their application for the load limit state evaluation. A summary of load factors and load combinations as applicable for a typical CIP cantilever wall is presented in Table 14.4-1 and **LRFD [3.4.1]**, respectively. Computed factored loads and moments are used for performing stability checks.

14.5.5 External Stability Checks

The external stability check includes checks for limiting eccentricity (overturning), bearing stress, and sliding at Strength I and Extreme Event II due to vehicle impact in cases where live load traffic is carried.

14.5.5.1 Eccentricity Check

The eccentricity of the retaining wall shall be evaluated in accordance with **LRFD [11.6.3.3]**. The location of the resultant force should be within 1/3 of base width of the foundation centroid (e<B/3) for foundations on soil, and within 0.45 of the base width of the foundation centroid (e<0.45B) for foundations on rock. If there is inadequate resistance to overturning (eccentricity value greater than limits given above), consideration should be given to either increasing the width of the wall base, or providing a deep foundation.

14.5.5.2 Bearing Resistance

The bearing resistance shall be evaluated at the strength limit state using factored loads and resistances. Bearing resistance of the walls founded directly on soil or rock shall be computed in accordance with 11.2 and **LRFD [10.6]**. The Bearing Resistance for walls on piles shall be computed in accordance with 11.3 and **LRFD [10.6]**. Figure 14.5-3 shows bearing stress criteria for a typical CIP wall on soil and rock respectively.

The vertical stress for footings on soil shall be calculated using:

$$\sigma_v = \frac{\sum V}{(B-2e)}$$

For walls founded on rock, the vertical stress is calculated assuming a linearly distributed pressure over an effective base area. The vertical stress for footings on rock shall be computed using:

$$\sigma_v = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right)$$

Where

ΣV = Summation of vertical forces

B = Base width

e = Eccentricity as shown in Figure 14.5-3 and Figure 14.5-4

If the resultant is outside the middle one-third of the wall base, then the vertical stress shall be computed using:

$$\sigma_{v\,max} = \left(\frac{2\sum V}{3\left(\frac{B}{2} - e\right)} \right)$$

$\sigma_{vmin} = 0$

The computed vertical stress shall be compared with factored bearing resistance in accordance with the **LRFD [10.6.3.1]** using following equation:

$q_r = \phi_b q_n > \sigma_v$

Where:

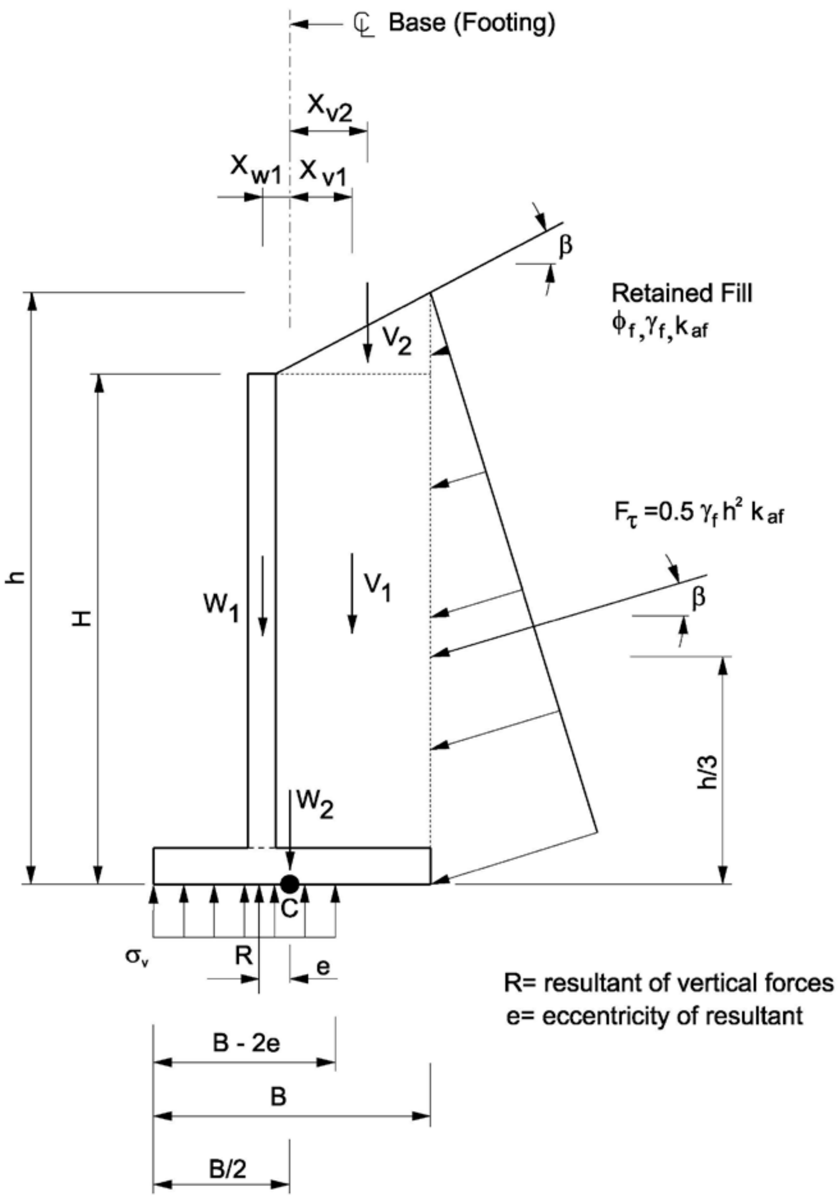
q_r = Factored bearing resistance

q_n = Nominal bearing resistance computed using **LRFD [10.6.3.1.2-a]**

σ_v = Vertical stress

B = Base width

e = Eccentricity as shown in Figure 14.5-3 and Figure 14.5-4

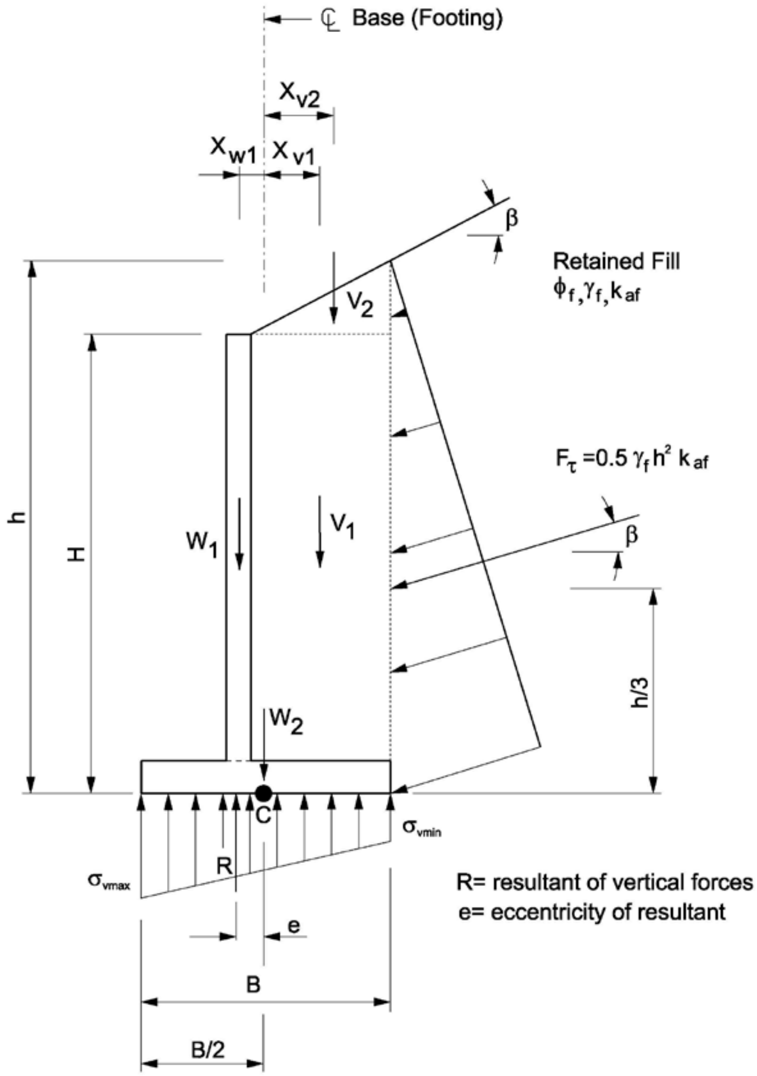


Summing Moments about Point C:

$$e = \frac{(F_T \cos\beta)h/3 - (F_T \sin\beta)B/2 - V_1 X_{V1} - V_2 X_{V2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 14.5-3

Loading Diagram and Bearing Stress Criteria for CIP Cantilever Walls on Soil (source *AASHTO LRFD*)



If e > B/6, σ_{vmin} will drop to zero, and as "e" increases, the portion of the heel of the footing which has zero vertical stress increases.

Summing Moments about Point C:

$$e = \frac{(F_T \cos\beta)h/3 - (F_T \sin\beta)B/2 - V_1 X_{V1} - V_2 X_{V2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 14.5-4

Loading Diagram and Bearing Stress Criteria for CIP Cantilever Walls on Rock
(source *AASHTO LRFD*)

14.5.5.3 Sliding

The sliding resistance of CIP cantilever walls is computed by considering the wall as a shallow footing resting on soil/rock or footing resting on piles in accordance with **LRFD [10.5]**. Sliding resistance of a footing resting on soil/rock foundation is computed in accordance with the **LRFD [10.6.3.4]** using the equation given below:

$$R_R = \phi\ R_n = \phi_\tau\ R_\tau + \phi_{ep} R_{ep}$$

Where:

R_R = Factored resistance against failure by sliding

R_n = Nominal sliding resistance against failure by sliding

ϕ_τ = Resistance factor for shear between soil and foundation per **LRFD [Table 10.5.5.2.2.1]**

R_τ = Nominal sliding resistance between soil and foundation

ϕ_{ep} = Resistance factor for passive resistance per **LRFD Table [10.5.5.2.2.1]**

R_{ep} = Nominal passive resistance of soil throughout the life of the structure

Contribution from passive earth pressure resistance against the embedded portion of the wall is neglected if the soil in front of the wall can be removed or weakened by scouring, erosion or any other means. Also, the live load surcharge is not considered as a stabilizing force over the heel of the wall when checking sliding.

If adequate sliding resistance cannot be achieved, footing design may be modified as follows:

- Increase the base width of the footing
- Construct a shear key
- Increase wall embedment to a sufficient depth, where passive resistance can be relied upon
- Incorporate a deep foundation, including battered piles (Usually a costly measure)

Guideline for selecting the shear key design is presented in 14.5.7.3. The design of wall footings resting on piles is performed in accordance with **LRFD [10.5]** and Chapter 11 - Foundation Support. Footings on piles resist sliding by the following:

1. Passive earth pressure in front of wall. Same as spread footing.
2. Lateral resistance of vertical piles as well as the horizontal components of battered piles. Maximum batter is 3 inches per foot. Refer to Chapter 11 - Foundation Support for lateral load capacity of piles.

3. Lateral resistance of battered or vertical piles in addition to horizontal component of battered piles. Refer to Chapter 11- Foundation Support for allowable lateral load capacity.

4. Do not use soil friction under the footing as consolidation of the soil may eliminate contact between the soil and footing.

14.5.5.4 Settlement

The settlement of CIP cantilever walls can be computed in accordance with guidelines and performance criteria presented in 14.4.7.2. The guideline for total and differential settlement is presented in Table 14.4-3. The actual performance limit can be changed for specific project requirements. For additional guidance contact the Geotechnical Engineering Unit.

14.5.6 Overall Stability

Investigate Service 1 load combination using an appropriate resistance factor and procedures discussed in **LRFD [11.6]** and 14.4.7.3. In general, the resistance factor, φ, may be taken as;

- 0.75 - where the geotechnical parameters are well defined, and slope does not support or contain a structural element.

- 0.65 – where the geotechnical parameters are based on limited information or the slope contains or supports a structural element.

14.5.7 Structural Resistance

The structural design of the stem and footing shall be performed in accordance with *AASHTO LRFD* and the design guidelines discussed below.

14.5.7.1 Stem Design

The initial sizing of the stem should be selected in accordance with criteria presented in 14.5.3. The stems of cantilever walls shall be designed as cantilevers supported at the footing. Axial loads (including the weight of the wall stem and frictional forces due to backfill acting on the wall stem) shall be considered in addition to the bending due to eccentric vertical loads, surcharge loads and lateral earth pressure if they control the design of the wall stems. The flexural design of the cantilever wall should be performed in accordance with *AASHTO LRFD*.

Loads from railings or parapets on top of the wall need not be applied simultaneously with live loads. These are dynamic loads which are resisted by the mass of the wall.

14.5.7.2 Footing Design

The footing of a cantilever wall shall be designed as a cantilever beam. The heel section must support the weight of the backfill soil and the shear component of the lateral earth pressure. All loads and moments must be factored using the criteria load factors discussed in 14.5.4. Use the following criteria when designing the footing.

1. Minimum footing thickness shall be selected in accordance with criteria presented in 14.5.3. The final footing thickness shall be based on shear at a vertical plane behind the stem.

2. For toe, design for shear at a distance from the face of the stem equal to the effective "d" distance of the footing. For heel, design for shear at the face of stem.

3. Where the footing is resting on piles, the piles shall be designed in accordance with criteria for pile design presented in Chapter 11 – Foundation Support. Embed piles six inches into footing. Place bar steel on top of the piles.

4. For spread footings, use a minimum of 3 inches clear cover at the bottom of footing. Use 2 inches clear cover for edge distance.

5. The critical sections for bending moments in footings shall be taken at the front and back faces of the wall stem. Bearing pressure along the bottom of the heel extension may conservatively be ignored. No bar steel is provided if the required area per foot is less than 0.05 square inches.

6. Design for heel moment, without considering the upward soil or pile reaction, is not required unless such a condition actually exists.

14.5.7.3 Shear Key Design

A shear key shall be provided to increase the sliding resistance when the factored sliding resistance determined using procedure discussed in 14.5.5.3 is inadequate. Use the following criteria when designing the shear key:

1. Place shear key in line with stem except under severe loading conditions.

2. The key width is 1'-0" in most cases. The minimum key depth is 1'-0".

3. Place shear key in unformed excavation against undisturbed material.

4. Analyze shear key in accordance with **LRFD [10.6.3.4]** and 14.5.5.3 .

5. The shape of shear key in rock is governed by the quality of the rock, but in general a 1 ft. by 1 ft key is appropriate.

14.5.7.4 Miscellaneous Design Information

1. Contraction joints shall be provided at intervals not exceeding 30 feet and expansion joints at intervals not exceeding 90 feet for reinforced concrete walls. Typical details of expansion and contraction joints are given in Figure 14.5-5. Expansion joints shall be constructed with a joint, filling material of the appropriate thickness to ensure the functioning of the joint and shall be provided with a waterstop capable of functioning over the anticipated range of joint movements.

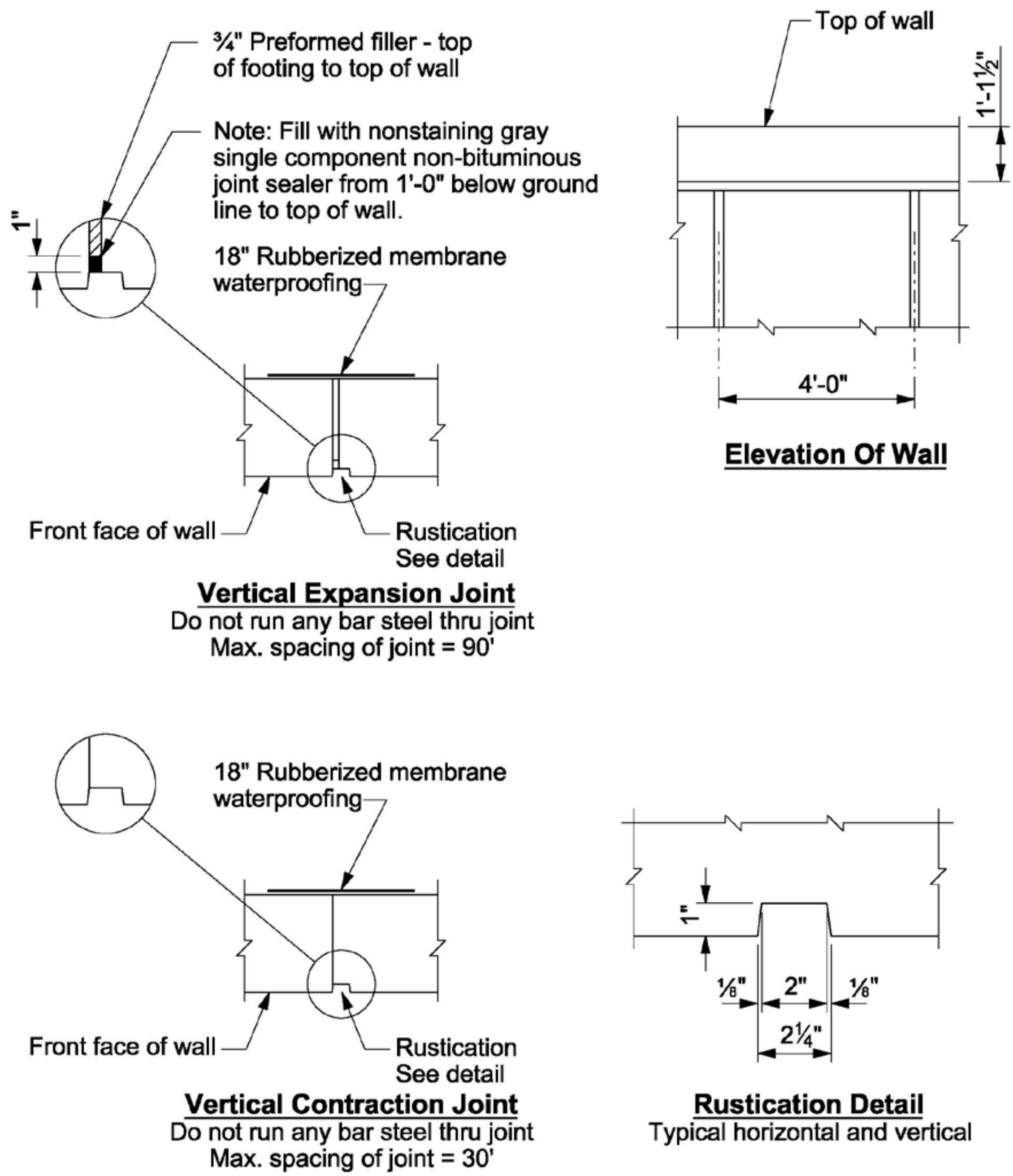


Figure 14.5-5
Retaining Wall Joint Details

2. Optional transverse construction joints are permitted in the footing, with a minimum spacing of three panel lengths. Footing joints should be offset a minimum of 1'-0 from wall joints. Run reinforcing bar steel thru footing joints.

3. The backfill material behind all cantilever walls shall be granular, free draining, non-expansive, non-corrosive material and shall be drained by weep holes with permeable material or other positive drainage systems, placed at suitable intervals and elevations. Structure backfill is placed behind the wall only to a vertical plane 18 inches beyond the face of footing. Lower limit is to the bottom of the footing.

4. If a wall is adjacent to a traveled roadway or sidewalk, use pipe underdrains in back of the wall instead of weep holes. Use a six-inch pipe wrapped underdrain located as detailed in this chapter. Provide a minimum slope of 0.5% and discharge to suitable drainage (i.e. a storm sewer system or ditch).

14.5.8 Design Tables for Cast-in-Place Concrete Cantilever Walls

Design tables suitable for use in preliminary design have been assembled and presented in this sub-section. These design tables are based on WisDOT design criteria and the material properties summarized in Table 14.5-1. Active earth pressure for the design tables was computed using the Rankine's equation for horizontal slopes and Coulomb's equation for surcharged slopes with the resultant perpendicular to the wall backface plus the wall friction angle. It was assumed that no water pressure exists. Service limit states were ignored in the analyses. The requirement of concrete is in accordance with **LRFD [5.4.2]** and 9.2. The requirement for bar steel is based on **LRFD [5.4.3]** and 9.3. The aforementioned assumptions were used in creating Table 14.5-2 thru Table 14.5-7. Refer to Figure 14.5-2 for details.

These tables should not be used if any of the assumptions or strength properties of the retained or foundation earth or the materials used for construction are different than those used in these design tables. The designer should also determine if the long-term or short-term soil strength parameters govern external stability analyses.

14.5.9 Design Examples

Refer to 14.18 for the design examples.

Design Criteria/Assumptions	Value
Concrete strength	3.5 ksi
Reinforcement yield strength	60 ksi
Concrete unit weight	150 pcf
Soil unit weight	120 pcf
Friction angle between fill and wall	21 degrees
Angle of Internal Friction (Soil - Backfill)	30 degrees

Angle of Internal Friction (Soil - Foundation)	34 degrees
Angle of Internal friction (Rock)	25 degrees
Cohesion (Soil)	0 psi
Cohesion (Rock)	20 psi
Soil Cover over Footing	4 feet
Stem Front Batter	0.25"/ft
Stem Back Batter	See Tables
Factored bearing resistance (On Soil)	**LRFD [10.6.3.1.2]**
Factored bearing resistance (On Rock)	20 ksf
Live Load Surcharge (Traffic)	240 psf
Live Load Surcharge (No Traffic)	100 psf
Lateral Earth Pressure (Horizontal Backfill)	Rankine
Lateral Earth Pressure (2:1 Backfill)	Coulomb

Table 14.5-1
Assumptions Summary for Preliminary Design of CIP Walls

HORIZONTAL BACKFILL – NO TRAFFIC – ON SOIL

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel		Shear	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa	Key	DSK
6	3'- 6"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	---	---	NO	---
8	4'- 6"	1'- 0"	1'- 6"	0	---	---	---	4	12	3' - 5"	4	12	NO	---
10	5'- 3"	1'- 3"	1'- 6"	0	---	---	---	4	12	3' - 10"	4	12	NO	---
12	6'- 3"	1'- 6"	2'- 0"	0	---	---	---	4	10	4' - 7"	5	12	NO	---
14	7'- 3"	1'- 9"	2'- 0"	0	4	12	2' - 7"	5	9	5' - 6"	6	10	NO	---
16	8'- 0"	2'- 0"	2'- 0"	0.50	4	12	2' - 10"	5	8	5' - 5"	6	10	NO	---
18	8'- 9"	2'- 3"	2'- 0"	0.50	4	12	3' - 1"	7	11	6' - 7"	6	8	NO	---
20	9'- 9"	2'- 6"	2'- 0"	0.50	4	10	3' - 4"	7	8	7' - 3"	7	8	NO	---
22	10'- 6"	2'- 9"	2'- 3"	0.50	4	9	3' - 7"	9	12	9' - 2"	9	12	NO	---
24	11'- 6"	3'- 0"	2'- 9"	0.50	4	9	3' - 10"	9	11	9' - 10"	8	9	NO	---
26	12'- 0"	4'- 0"	2'- 9"	0.50	5	8	4' - 10"	8	8	8' - 5"	8	8	YES	1'- 6"
28	13'- 0"	5'- 0"	3'- 0"	0.75	7	11	6' - 6"	8	8	7' - 9"	8	7	YES	1'- 6"

Table 14.5-2
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – TRAFFIC – ON SOIL

H	B	A	D	Batter	Toe Steel			Heel Steel			Stem Steel		Shear	
(ft)	(ft)	(ft)	(ft)	(in/ft)	Size	Spa	L	Size	Spa	L	Size	Spa	Key	DSK
6	4'- 6"	0'- 6"	1'- 6"	0	---	---	---	4	12	3' - 11"	---	---	NO	---
8	5'- 3"	0'- 9"	1'- 6"	0	---	---	---	4	11	4' - 5"	4	12	NO	---
10	6'- 6"	1'- 0"	1'- 6"	0	---	---	---	6	12	5' - 11"	4	8	NO	---
12	7'- 3"	1'- 3"	2'- 0"	0	---	---	---	6	11	6' - 5"	5	9	NO	---
14	8'- 3"	1'- 6"	2'- 0"	0	---	---	---	7	10	7' - 7"	6	9	NO	---
16	9'- 0"	2'- 3"	2'- 0"	0.50	4	12	3'- 1"	7	10	7'- 0 "	6	9	NO	---
18	9'- 3"	2'- 9"	2'- 0"	0.50	4	10	3'- 7"	7	10	6' - 7"	8	12	YES	1'- 0"
20	10'- 0"	3'- 6"	2'- 0"	0.50	5	9	4'- 4"	6	7	6'- 0 "	8	10	YES	1'- 0"
22	11'- 0"	4'- 3"	2'- 3"	0.50	5	7	5'- 1"	6	7	6' - 2"	7	7	YES	1'- 0"
24	11'- 9"	5'- 0"	2'- 6"	0.50	7	10	6'- 6"	6	7	6'- 0 "	9	11	YES	1'- 6"
26	12'- 9"	5'- 9"	2'- 9"	0.50	8	11	7'- 9"	6	7	6' - 2"	9	9	YES	1'- 6"
28	14'- 3"	7'- 0"	3'- 0"	0.75	9	11	9'- 7"	6	7	5' - 9"	9	9	YES	2'- 0"

Table 14.5-3
Reinforcement for Cantilever Retaining Walls

2:1 BACKFILL – NO TRAFFIC – ON SOIL

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa	Shear Key	DSK
6	4'- 6"	2'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12	YES	1'- 0"
8	6'- 0"	2'- 6"	1'- 6"	0	4	12	3'- 4"	4	12	3' - 5"	4	9	YES	1'- 0"
10	7'- 6"	2'- 0"	1'- 6"	0	4	12	2'- 10"	6	11	5' - 11"	6	9	YES	1'- 0"
12	9'- 0"	1'- 9"	2'- 0"	0	4	12	2'- 7"	7	9	8' - 2"	8	11	YES	1'- 0"
14	10'- 6"	2'- 6"	2'- 6"	0	4	12	3'- 4"	8	10	9' - 8"	9	10	YES	1'- 6"
16	12'- 3"	3'- 9"	2'- 9"	0.50	5	12	4'- 7"	7	7	8' - 10"	9	10	YES	2'- 0"
18	14'- 0"	4'- 6"	3'- 0"	0.50	6	12	5'- 7"	9	9	11' - 2"	10	10	YES	2'- 0"
20	15'- 6"	5'- 6"	3'- 3"	0.50	7	11	7'- 0"	10	11	12' - 8"	10	8	YES	2'- 9"

Table 14.5-4
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – NO TRAFFIC – ON ROCK

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa
6	2'- 9"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	4	12
8	3'- 6"	1'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12
10	4'- 3"	1'- 3"	1'- 6"	0	---	---	---	4	12	2' - 10"	4	12
12	5'- 0"	1'- 6"	2'- 0"	0	4	12	2'- 4"	4	12	3' - 4"	5	12
14	5'- 9"	1'- 9"	2'- 0"	0	4	12	2'- 7"	4	12	3' - 10"	6	10
16	6'- 6"	2'- 0"	2'- 0"	0.50	4	12	2'- 10"	4	11	3' - 8"	6	10
18	7'- 3"	2'- 3"	2'- 0"	0.50	4	11	3'- 1"	5	12	4' - 3"	6	8
20	7'- 9"	2'- 6"	2'- 0"	0.50	5	11	3'- 4"	5	9	4' - 5"	8	11
22	8'- 6"	2'- 9"	2'- 0"	0.50	5	9	3'- 7"	6	10	5' - 1"	7	7
24	9'- 3"	3'- 0"	2'- 0"	0.50	6	10	4'- 1"	7	10	6'- 0 "	9	11
26	10'- 0"	3'- 3"	2'- 3"	0.50	6	9	4'- 4"	8	11	7' - 2"	10	12
28	10'- 6"	3'- 6"	2'- 6"	0.75	6	8	4'- 7"	8	11	6' - 9"	9	9

Table 14.5-5
Reinforcement for Cantilever Retaining Walls

HORIZONTAL BACKFILL – TRAFFIC – ON ROCK

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa
6	3'- 6"	0'- 9"	1'- 6"	0	---	---	---	---	---	---	4	12
8	4'- 3"	1'- 0"	1'- 6"	0	---	---	---	4	12	3' - 2"	4	12
10	5'- 0"	1'- 3"	1'- 6"	0	---	---	---	4	12	3' - 7"	4	8
12	5'- 9"	1'- 6"	2'- 0"	0	---	---	---	4	12	4' - 1"	5	9
14	6'- 6"	1'- 9"	2'- 0"	0	4	12	2'- 7"	4	8	4' - 6"	6	9
16	7'- 3"	2'- 0"	2'- 0"	0.50	4	12	2'- 10"	4	7	4' - 5"	7	12
18	8'- 0"	2'- 3"	2'- 0"	0.50	4	11	3'- 1"	6	11	5' - 4"	8	12
20	8'- 9"	2'- 6"	2'- 3"	0.50	4	9	3'- 4"	6	9	5' - 9"	8	10
22	9'- 6"	2'- 9"	2'- 6"	0.50	5	12	3'- 7"	7	11	6' - 8"	9	12
24	10'- 3"	3'- 0"	2'- 9"	0.50	5	10	3'- 10"	7	9	7' - 1"	9	11
26	11'- 0"	4'- 0"	2'- 6"	0.50	7	10	5'- 6"	8	11	7' - 5"	8	7
28	11'- 9"	4'- 3"	2'- 9"	0.75	6	7	5'- 4"	8	11	7' - 3"	8	7

Table 14.5-6
Reinforcement for Cantilever Retaining Walls

2:1 BACKFILL – NO TRAFFIC – ON ROCK

H (ft)	B (ft)	A (ft)	D (ft)	Batter (in/ft)	Toe Steel Size	Toe Steel Spa	Toe Steel L	Heel Steel Size	Heel Steel Spa	Heel Steel L	Stem Steel Size	Stem Steel Spa
6	3'- 9"	2'- 0"	1'- 6"	0	---	---	---	---	---	---	4	12
8	5'- 0"	2'- 9"	1'- 6"	0	4	12	3'- 7"	4	12	2' - 2"	4	12
10	6'- 0"	3'- 3"	1'- 6"	0	4	9	4'- 1"	4	12	2' - 7"	6	12
12	7'- 0"	4'- 0"	2'- 0"	0	5	11	4'- 10"	4	12	2' - 10"	6	9
14	8'- 3"	4'- 6"	2'- 0"	0	6	10	5'- 7"	4	12	3' - 7"	8	11
16	9'- 0"	5'- 3"	2'- 0"	0.50	8	11	7'- 3"	4	12	2' - 11"	8	11
18	10'- 0"	4'- 9"	2'- 0"	0.50	8	10	6'- 9"	6	11	4' - 10"	9	10
20	11'- 3"	4'- 0"	2'- 6"	0.50	7	10	5'- 6"	8	10	8'- 0 "	11	11
22	12'- 3"	4'- 6"	3'- 0"	0.50	7	9	6'- 0"	9	12	9' - 2"	11	9

Table 14.5-7
Reinforcement for Cantilever Retaining Walls

14.5.10 Summary of Design Requirements

1. Stability Check

   a. Strength I and Extreme Event II limit states

      - Eccentricity
      - Bearing Stress
      - Sliding

   b. Service I limit states

      - Overall Stability
      - Settlement

2. Foundation Design Parameters

   Use values provided by Geotechnical analysis

3. Concrete Design Data

   - f'c = 3500 psi
   - fy = 60,000 psi

4. Retained Soil

   - Unit weight = 120 lb/ft^3
   - Angle of internal friction - use value provided by Geotechnical analysis

5. Soil Pressure Theory

   - Coulomb theory for short heels or Rankine theory for long heels at the discretion of the designer.

6. Surcharge Load

   - Traffic live load surcharge = 2 feet = 240 lb/ft^2
   - If no traffic surcharge, use 100 lb/ft^2

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength I-a	0.90	1.00	1.75	1.75	1.50		Sliding, eccentricity
Strength I-b	1.25	1.35	1.75	1.75	1.50		Bearing /wall strength
Extreme II-a	0.90	1.00	-	-	-	1.00	Sliding, eccentricity
Extreme II-b	1.25	1.35	-	-	-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

<u>Table 14.5-8</u>
Load Factor Summary for CIP Walls

8. Bearing Resistance Factors

- ϕ_b = 0.55 **LRFD [Table 11.5.7-1]**

9. Sliding Resistance Factors

- ϕ_τ = 1.0 **LRFD [Table 11.5.7-1]**
- ϕ_{ep} = 0.5 **LRFD Table [10.5.5.2.2-1]**

14.6 Mechanically Stabilized Earth Retaining Walls

14.6.1 General Considerations

Mechanically Stabilized Earth (MSE) is the term used to describe the practice of reinforcing a mass of soil with either metallic or geosynthetic soil reinforcement which allows the mass of soil to function as a gravity retaining wall structure. The soil reinforcement is placed horizontally across potential planes of shear failure and develops tension stresses to keep the soil mass intact. The soil reinforcement is attached to a wall facing located at the front face of the wall.

The design of MSE walls shall meet the *AASHTO LRFD* requirements in accordance with 14.4.2. The service life requirement for both permanent and temporary MSE wall systems is presented in 14.4.3.

The MSE walls shall be designed for external stability of the wall system and internal stability of the reinforced soil mass. The global stability shall also be considered as part of design evaluation. MSE walls are proprietary wall systems and the design responsibilities with respect to global, external, and internal stability as well as settlement are shared between the designer (WisDOT or Consultant) and contractor. The designer is responsible for the overall stability, preliminary external stability and settlement whereas the contractor is responsible for the internal stability, compound stability and structural design of the wall. The responsibilities of the designer and contractor are outlined in 14.6.3.2. The design and drawings of MSE walls provided by the contractor must also be in compliance with the WisDOT special provisions as stated in 14.15.2 and 14.16

The guidelines provided herein for MSE walls do not apply to geometrically complex MSE wall systems such as tiered walls (walls stacked on top of one another), back-to-back walls, or walls which have trapezoidal sections. Design guidelines for these cases are provided in publications FHWA-NHI-10-024 and FHWA-NHI-10-025.

Horizontal alignment and grades at the bottom and top of the wall are determined by the design engineer. The design must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in the *Bridge Manual* and *FDM*.

14.6.1.1 Usage Restrictions for MSE Walls

Construction of MSE walls with either block or panel facings should not be used when any of the following conditions exist:

1. If the available construction limit behind the wall does not meet the soil reinforcement length requirements.

2. Sites where extensive excavation is required or sites that lack granular soils and the cost of importing suitable fill material may render the system uneconomical.

3. At locations where erosion or scour may undermine or erode the reinforced fill zone or any supporting leveling pad.

4. Soil is contaminated by corrosive material such as acid mine, drainage, other industrial pollutants, or any other condition which increases corrosion rate, such as the presence of stray electrical currents.

5. There is potential for placing buried utilities within (or below) the reinforced zone unless access is provided to utilities without disrupting reinforcement and breakage or rupture of utility lines will not have a detrimental effect on the stability of the wall. Contact WisDOT's Structures Design Section.

14.6.2 Structural Components

The main structural elements or components of an MSE wall are discussed below. General elements of a typical MSE wall are shown in Figure 14.6-1. These include:

- Selected Earthfill in the Reinforced Earth Zone
- Reinforcement
- Wall Facing Element
- Leveling Pad
- Wall Drainage

A combination of different wall facings and reinforcement provide a choice of selecting an MSE wall which can be used for several different functions.

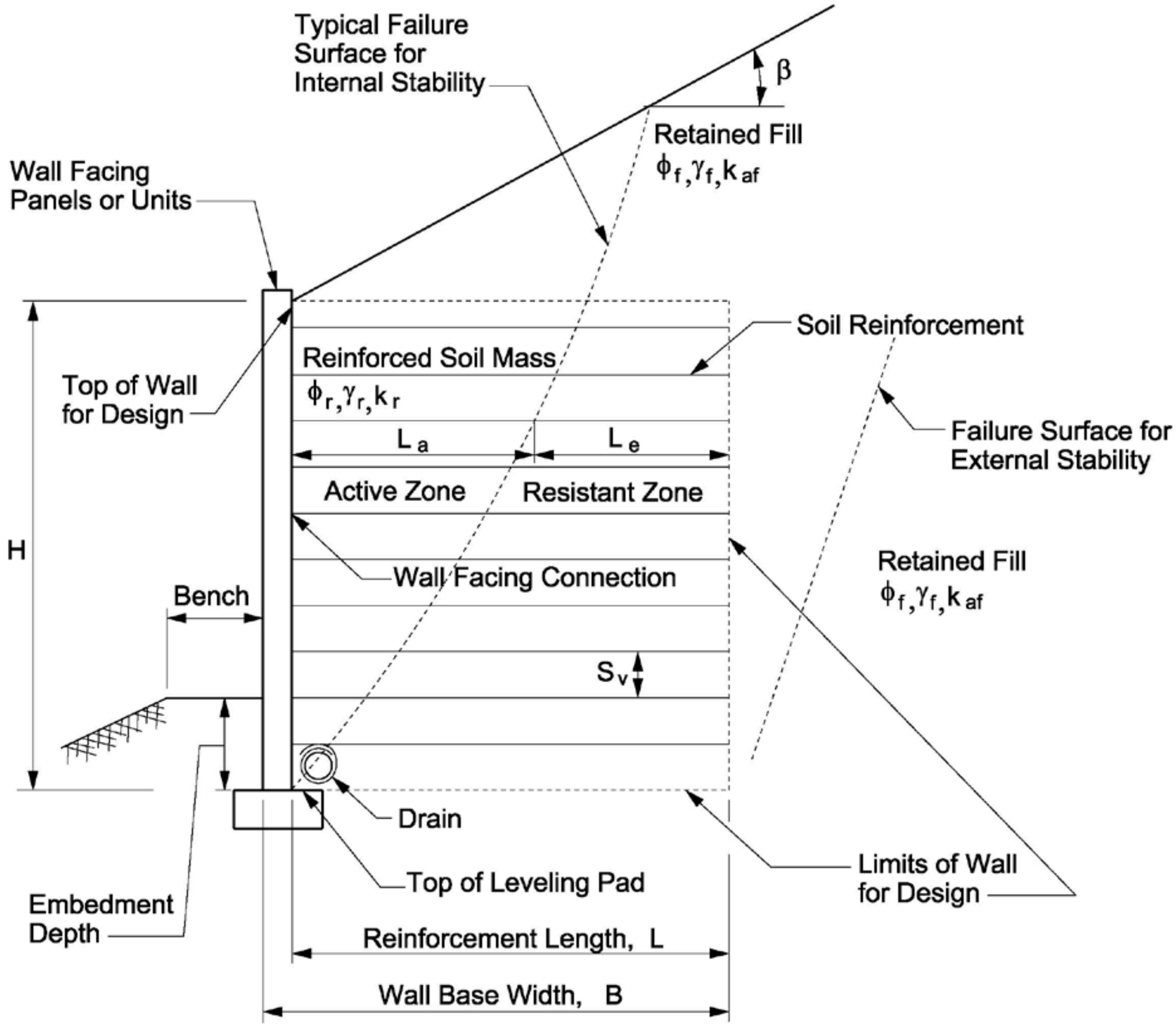


Figure 14.6-1
Structural Components of MSE Walls

14.6.2.1 Reinforced Earthfill Zone

The reinforced backfill to be used to construct the MSE wall shall meet the criteria in the wall specifications. The backfill shall be free from organics, or other deleterious material. It shall not contain foundry sand, bottom ash, blast furnace slag, or other potentially corrosive material. It shall meet the electrochemical criteria given in Table 14.6-1.

Reinforcement Material	Property	Criteria
Metallic	Resistivity	> 3000 ohm-cm
Metallic	Chlorides	< 100 ppm
Metallic	Sulfates	< 200 ppm
Metallic	pH	5.0 < pH < 10.0
Geosynthetic	pH	4.5 < pH < 9.0
Metallic/Geosynthetic	Organic Content	< 1.0 %

Table 14.6-1
Electrochemical Properties of Reinforced Fill MSE Walls

An angle of internal friction of 30 degrees and unit weight of 120 pcf shall be used for the stability analyses as stated in 14.4.6. If it is desired to use an angle of internal friction greater than 30 degrees, it shall be determined by the most current wall specifications.

14.6.2.2 Reinforcement:

Soil reinforcement can be either metallic (strips or bar grids like welded wire fabric) or non-metallic including geotextile and geogrids made from polyester, polypropylene, or high density polyethylene. Metallic reinforcements are also known as inextensible reinforcement and the non-metallic as extensible. Inextensible reinforcement deforms less than the compacted soil infill used in MSE walls, whereas extensible reinforcement deforms more than compacted soil infill

The metallic or inextensible reinforcement is mild steel, and usually galvanized or epoxy coated. Three types of steel reinforcement are typically used:

Steel Strips: The steel strip type reinforcement is mostly used with segmental concrete facings. Commercially available strips are ribbed top and bottom, 2 to 4 inch wide and 1/8 to 5/32 inch thick.

Steel grids: Welded wire steel grids using two to six W7.5 to W24 longitudinal wires spaced either at 6 or 8 inches. The transverse wire may vary from W11 to W20 and are spaced from 9 to 24 inches apart.

Welded wire mesh: Welded wire meshes spaced at 2 by 2 inch of thinner steel wire can also be used.

The galvanized steel reinforcement that is used for soil reinforcement is oversized in cross sectional areas to account for the corrosion that occurs during the life of the structure and the resulting loss of section. The net section remaining after corrosion at the end of the design service life is used to check design requirements

The non-metallic or extensible reinforcement includes the following:

Geogrids: The geogrids are mostly used with modular block walls.

Geotextile Reinforcement: High strength geotextile can be used principally with wrap-around and temporary wall construction.

Corrosion of the wall anchors that connect the soil reinforcement to the wall face must also be accounted for in the design.

14.6.2.3 Facing Elements

The types of facings element used in the different MSE walls mainly control aesthetics, provide protection against backfill sloughing and erosion, and may provide a drainage path in certain cases. A combination of different wall facings and reinforcement provide a choice of selecting an MSE wall which can be used for several different functions.

Major facing types are:

- Segmental precast concrete panels
- Dry cast or wet cast modular blocks
- Full height pre-cast concrete panels (tilt-up)
- Cast-in-place concrete facing
- Geotextile-reinforced wrapped face
- Geosynthetic /Geogrid facings
- Welded wire grids

Segmental Precast Concrete Panels

Segmental precast concrete panels include small panels (<30 sq ft) to larger (>30 sq ft) with a minimum thickness of 5-½ inches and are of a square, rectangular, cruciform, diamond, or hexagonal geometry. The geometric pattern of the joints and the smooth uniform surface finish of the factory provided precast panels give an aesthetically pleasing appearance. Segmental precast concrete panels are proprietary wall components.

Wall panels are available in a plain concrete finish or numerous form liner finishes and textures. An exposed aggregate finish is also available along with earth tone colors. Although color can be obtained by adding additives to the concrete mix it is more desirable to obtain color by applying concrete stain and/or paint at the job site. Aesthetics do affect wall costs.

WisDOT requires that MSE walls utilize precast concrete panels when supporting traffic live loads which are in close proximity to the wall. Panels are also allowed as components of an abutment structure. Either steel strips or welded wire fabric is allowed for soil reinforcement when precast concrete panels are used as facing of the MSE wall system.

Walls with curved alignments shall limit radii to 50 feet for 5 feet wide panels and 100 feet for 10 feet wide panels. Typical joint openings are not suitable for wall alignments following a tighter curve. Special joints or special panels that are less than 5 feet wide may be able to accommodate tighter curves. In general, MSE wall structures with panel type facings shall be limited to wall heights of 33 feet. Contact Structures Design Section for approval on case by case basis.

Concrete Modular Blocks Facings

Concrete modular block retaining walls are constructed from modular blocks typically weighing from 40 to 100 pounds each, although blocks over 200 pounds are rarely used. Nominal front to back width ranges between 8 to 24 inches. Modular blocks are available in a large variety of facial textures and colors providing a variety of aesthetic appearances. The shape of the blocks usually allows the walls to be built along a curve, either concave (inside radius) or convex (outside radius). The blocks or units are dry stacked meaning mortar or grout is not used to bond the units together except for the top two layers. Figure 14.6-2 shows various types of blocks available commercially. Figure 14.6-3 shows a typical modular block MSE wall system along with other wall components. Most modular block MSE walls are reinforced with geogrids.

Modular blocks can be either dry cast or wet cast. Dry cast (small) blocks are mass produced by using a zero slump concrete that allows forms to be stripped faster than wet cast (large) blocks. MSE walls usually use dry cast blocks since they are usually a cheaper facing and wall stability is provided by the reinforced mass. Gravity walls rely on facing size and mass for wall stability. For minor walls dry cast blocks are typically used and for taller gravity walls wider wet cast blocks are normally required to satisfy stability requirements.

Concrete modular blocks are proprietary wall component systems. Each proprietary system has its own unique method of locking the units together to resist the horizontal shear forces that develop. Fiberglass pins, stainless steel pins, glass filled nylon clips and mechanical interlocking surfaces are some of the methods utilized. Any pins or hardware must be manufactured from corrosion resistant materials.

During construction of these systems, the voids are filled with granular material such as crushed stone or gravel. Most of the systems have a built in or automatic set-back (incline angle of face to the vertical) which is different for each proprietary system. Blocks used on WisDOT projects must be of one piece construction. A minimum weight per block or depth of block (distance measured perpendicular to wall face) is not specified on WisDOT projects. The minimum thickness allowed of the front face is 4 inches (measured perpendicular from the front face to inside voids greater than 4 square inches). Also the minimum allowed thickness of any other portions of the block (interior walls or exterior tabs, etc.) is 2 inches.

Alignments that are not straight (i.e. kinked or curved) shall use 90 degree corners or curves. The minimum radius should be limited to 8 feet. For a concave wall the radius is measured to the front face of the bottom course. For convex walls the radius is measured to the front face of the top course. In no case shall the radius be less than 6 feet. It is WisDOT policy to design modular block MSE walls for a maximum height of 22 ft (measured from the top of the leveling pad to the top of the wall).

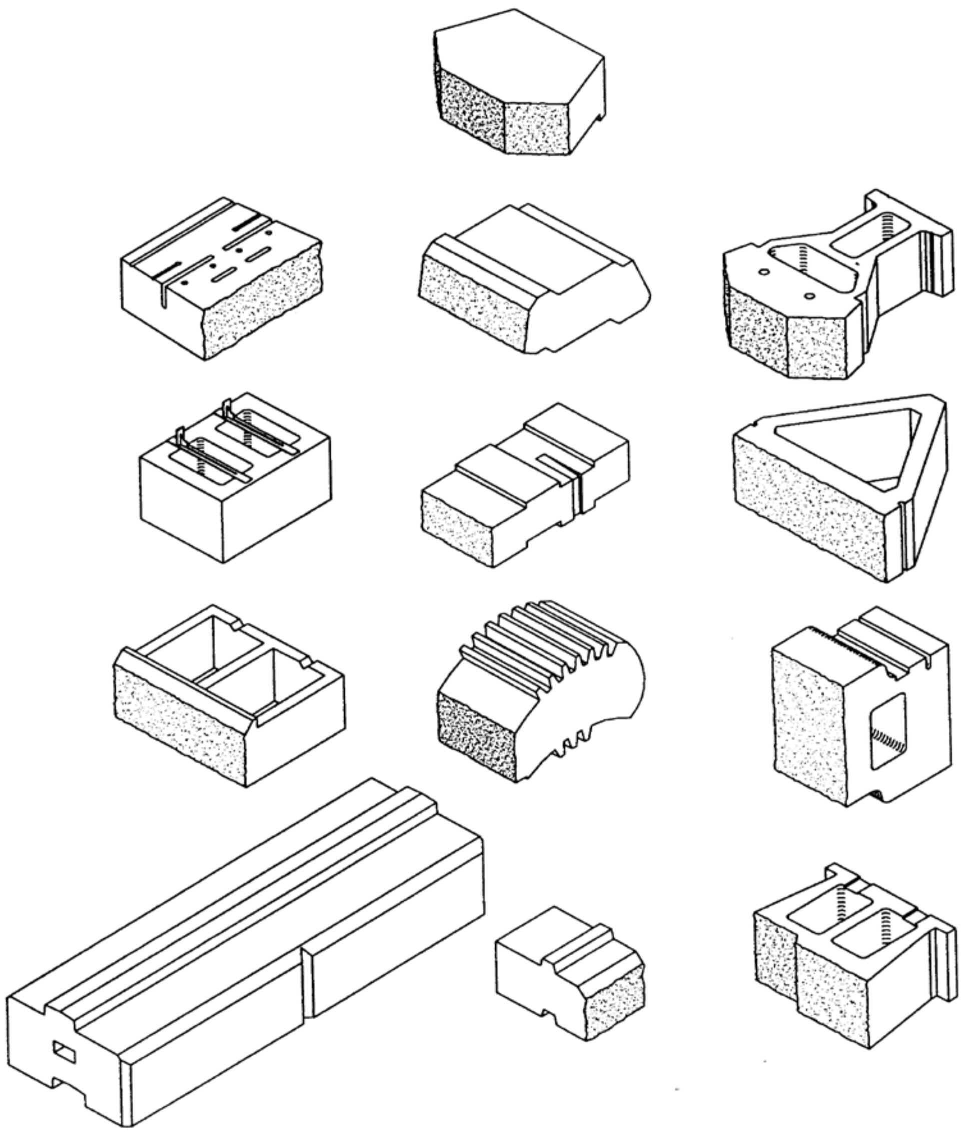
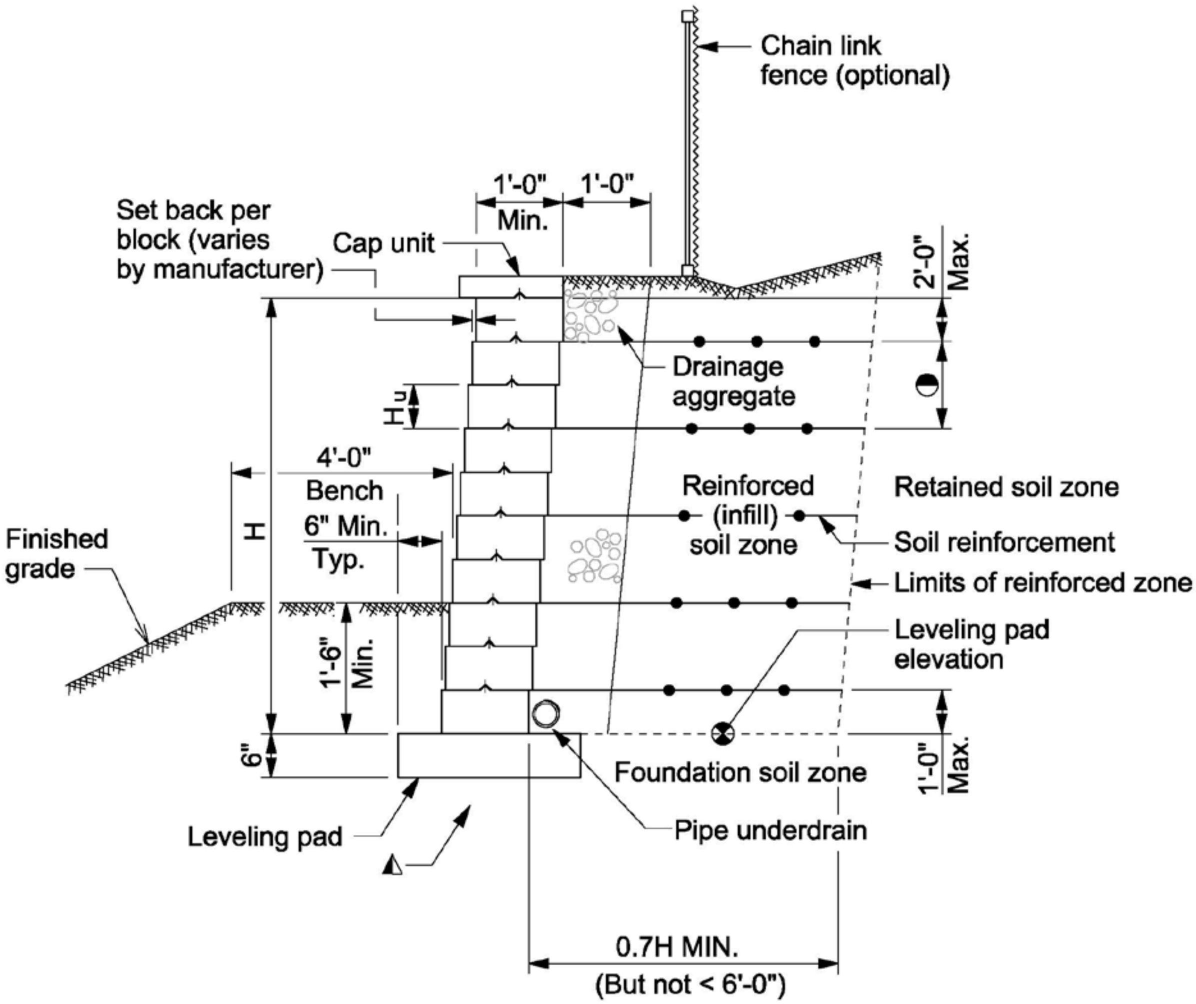


Figure 14.6-2
Modular Blocks
(Source FHWA-NHI-10-025)



Modular Block MSE Wall

◭ Ground improvement meaures should be taken when the soil below the levelling pad is poor or subject to frost heave.

◑ Maximum vertical spacing of soil reinforcement layers shall be two times the block depth (Hu) or 32 inches, whichever is less.

Figure 14.6-3
Typical Modular Block MSE Walls

MSE Wire-Faced Facing

Welded wire fabric facing is used to build MSE wire-faced walls. These are essentially MSE walls with a welded wire fabric facing instead of a precast concrete facing. The wire size, spacing and patterns used in the facing are developed from performance data of full size wall tests and from applications in actual walls. A test to determine the connection strength between the soil reinforcement and the facing panels is required. Some systems do not use a connection because the ground reinforcement and facing panel are of one piece construction.

MSE wire-faced wall systems usually incorporate a backing mat behind the front facing. A fine metallic screen and geotextile fabric is placed behind the backing mat (or behind the facing if a backing mat is not used) to prevent the backfill from passing thru the front face.

MSE wire-faced walls can tolerate considerable differential settlement because of the flexibility of the wire facing. The limiting differential settlement is 1/50. The flexibility of the wire facing results in face bulging between ground reinforcement. The actual amount varies per system but normally is less than one inch. Recommended limits on bulging are 2" for permanent walls and 3" for temporary walls. This type of wall works well when a permanent wall facing can be placed after settlement/movement has occurred.

When MSE wire-faced walls are used for permanent wall applications, all steel components must be galvanized. When used for temporary wall applications black steel (non-galvanized) may be used since the walls are usually left in place and buried.

Temporary MSE wire-faced walls can be used as temporary shoring if site conditions permit. This wall type can also be used when staged construction is required to maintain traffic when an existing roadway is being raised and/or widened in conjunction with bridge approaches, railroad crossings or road reconstruction.

Cast-In- Place Concrete Facing

MSE walls with cast in place concrete facings are identical to MSE wire faced walls except a cast-in-place concrete facing is added after the wire face wall is erected. Modifications are made to the standard wire face wall detail to anchor the concrete facing to the wire facing and soil reinforcement. They are usually used when a special aesthetic facial treatment is required without the numerous joints that are common to precast panels. They can also be used where differential or total settlement is above tolerable limits for other wall types. A MSE wire faced wall can be constructed and allowed to settle with the concrete facing added after consolidation of the foundation soils has occurred.

The cast-in-place concrete facing shall be a minimum of 8-inches thick and contain coated or galvanized reinforcing steel. This is required because the panels and/or anchor that extend into the cast-in-place concrete are galvanized and a corrosion cell would be created if black steel contacts galvanized steel. All wire ties and bar chairs used in the cast–in-place concrete must also be coated or galvanized. Note that the 8-inch minimum wall thickness will occur at the points of maximum panel bulging and that the wall will be thicker at other locations. Also note that the 8-inch minimum is measured from the trough of any form liner or rustication.

Vertical construction joints are required in the cast-in-place concrete facing to allow for expansion and contraction and to allow for some differential settlement. Closer spacing of vertical construction joints is required when differential settlement may occur, but by delaying the placement of the cast-in-place concrete, the effects of differential settlement is minimized. Higher walls also require closer spacing of vertical construction joints if differential settlement is anticipated. Horizontal construction joints may disrupt the flow of a special aesthetic facial treatment and are sometimes not allowed for that reason. The designer should specify if optional horizontal construction joints are allowed. Cork filler is placed at vertical construction joints because cork is compressible and will allow some expansion and rotation to occur at the joint. An expandable polyvinyl chloride waterstop (PCW) is used on the back side of a vertical construction joint. Since forms are only used at the front face of the wall the PCW can be attached to a 10-inch board which is supported by the wire facing. The 8-inch minimum wall thickness may be decreased at the location of the vertical construction joint to accommodate the PCW and its support board.

Geosynthetic Facing

Geosynthetic reinforcements are looped around at the facing to form the exposed face of the MSE Wall. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire. Geogrid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. This facing is generally used in temporary applications. Similar to wire faced walls, these walls typically have a geotextile behind the geogrids, to prevent material from passing through the face.

14.6.3 Design Procedure

14.6.3.1 General Design Requirements

The procedure for design of an MSE wall requires evaluation of external stability and internal stability (structural design) at Strength Limit States and overall stability and vertical/lateral movement at Service Limit State. The Extreme Event II load combination is used to design and analyze for vehicle impact where traffic barriers are provided to protect MSE walls. The design and stability is performed in accordance with *AASHTO LRFD* and design guidance discussed in 14.4.

14.6.3.2 Design Responsibilities

MSE walls are proprietary wall systems and the structural design of the wall system is provided by the contractor. The structural design of the MSE wall system must include an analysis of internal stability (soil reinforcement pullout and stress) and local stability (facing connection forces and internal panel stresses). Additionally, the contractor should also provide internal drainage. Design drawings and calculations must be submitted to the Bureau of Structures for acceptance.

External stability, overall stability and settlement calculations are the responsibility of the WISDOT/Consultant designer. Compound stability is the responsibility of the Contractor. Soil borings and soil design parameters are provided by Geotechnical Engineer.

Although abutment loads can be supported on spread footings within the reinforced soil zone, it is WisDOT policy to support the abutment loads for multiple span structures on piles or shafts that pass through the reinforced soil zone to the in-situ soil below. Piles shall be driven prior to the placement of the reinforced earth. Strip type reinforcement can be skewed around the piles but must be connected to the wall panels and must extend to the rear of the reinforced soil zone.

For continuous welded wire fabric reinforcement, the contractor should provide details on the plans showing how to place the reinforcement around piles or any other obstacle. Abutments for single span structures may be supported by spread footings placed within the soil reinforcing zone, with WISDOT's approval. Loads from such footings must be considered for both internal wall design and external stability considerations.

14.6.3.3 Design Steps

Design steps specific to MSE walls are described in FHWA publication No*. FHWA-NHI-10-24* and modified shown below:

1. Establish project requirements including all geometry, loading conditions (transient and/or permanent), performance criteria, and construction constraints.
2. Evaluate existing topography, site subsurface conditions, in-situ soil/rock properties, and wall backfill parameters.
3. Select MSE wall using project requirement per step 1 and wall selection criteria discussed in 14.3.1.
4. Based on initial wall geometry, estimate wall embedment depth and length of reinforcement.
5. Estimate unfactored loads including earth pressure for traffic surcharge or sloping back slope and /or front slope.
6. Summarize load factors, load combinations, and resistance factors
7. Calculate factored loads for all appropriate limit states and evaluate (external stability) at Strength I Limit State
    a. sliding
    b. eccentricity
    c. bearing
8. Compute settlement at Service limit states
9. Compute overall stability at Service limit states
10. Compute vertical and lateral movement
11. Design wall surface drainage systems
12. Compute internal stability
    a. Select reinforcement
    b. Estimate critical failure surface
    c. Define unfactored loads
    d. Calculate factored horizontal stress and maximum tension at each reinforcement level
    e. Calculate factored tensile stress in each reinforcement
    f. Check factored reinforcement pullout resistance
    g. Check connection resistance requirements at facing
13. Design facing element
14. Design subsurface drainage

Steps 1-11 are completed by the designer and steps 12-14 are completed by the contractor after letting.

14.6.3.4 Initial Geometry

Figure 14.6-1 provides MSE wall elements and dimensions that should be established before making stability computations for the design of an MSE wall. The height (H) of an MSE wall is measured vertically from the top of the MSE wall to the top of the leveling pad. The length of reinforcement (L) is measured from the back of MSE wall panels. Alternately, the length of reinforcement (L1) is measured from the front face for modular block type MSE walls.

The MSE walls, with panel type facings, generally do not exceed heights of 35 feet, and with modular block type facings, should not exceed heights of 22 feet. Wall heights in excess of these limits will require approval on a case by case basis from WisDOT.

In general, a minimum reinforcement length of 0.7H or 8 feet whichever is greater shall be provided. MSE wall structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcement lengths of 0.8H to 1.1H. As an exception, a minimum reinforcement length of 6.0 feet or 0.7H may be provided in accordance with **LRFD [C11.10.6.2.1]** provided all conditions for external and internal stability are met and smaller compaction equipment is used on a case by case basis as approved by WisDOT. MSE walls may be built to heights mentioned above; however, the external stability requirements may limit MSE wall height due to bearing capacity, settlement, or stability problems.

14.6.3.4.1 Wall Embedment

The minimum wall embedment depth to the bottom of the MSE wall reinforced backfill zone (top of the leveling pad shown in **LRFD [Figure 11.10.2-1]** and Figure 14.6-1 shall be based on external stability analysis (sliding, bearing resistance, overturning, and settlement) and the global (overall) stability requirements.

Minimum MSE wall leveling pad (and front face) embedment depths below lowest adjacent grade in front of the wall shall be in accordance with **LRFD [11.10.2.2]**, including the minimum embedment depths indicated in **LRFD [Table C11.10.2.2-1]** or 1.5 ft. whichever is greater. The embedment depth of MSE walls along streams and rivers shall be at least 2.0 ft below the potential scour elevation in accordance with **LRFD [11.10.2.2]** and the *Bridge Manual*.

WisDOT policy item:

The minimum depth of embedment of MSE walls shall be 1.5 feet

14.6.3.4.2 Wall Backslopes and Foreslopes

The wall backslopes and foreslopes shall be designed in accordance with 14.4.5.4.4. A minimum horizontal bench width of 4 ft (measured from bottom of wall horizontally to the slope face) shall be provided, whenever possible, in front of walls founded on slopes. This minimum bench width is required to protect against local instability near the toe of the wall.

14.6.3.5 External Stability

The external stability of the MSE walls shall be evaluated for sliding, limiting eccentricity, and bearing resistance at the Strength I limit state. The settlement shall be calculated at Service I limit state.

Unfactored loads and factored load shall be developed in accordance with 14.6.3.5.1. It is assumed that the reinforced mass zone acts as a rigid body and that wall facing, the reinforced soil and reinforcement act as a rigid body.

For adequate stability, the goal is to have the factored resistance greater than the factored loads. According to publication FHWA-NHI-10-024, a capacity to demand ratio (CDR) can be used to quantify the factored resistance and factored load. CDR has been used to express the safety of the wall against sliding, limiting eccentricity, and bearing resistance.

14.6.3.5.1 Unfactored and Factored Loads

Unfactored loads and moments are computed based on initial wall geometry and using procedures defined in 14.4.5.4.5. The loading diagrams for one of the 3 possible earth pressure conditions are developed. These include 1) horizontal backslope with traffic surcharge shown in Figure 14.4-2; 2) sloping backslope shown in Figure 14.4-3; and, 3) broken backslope condition as shown in Figure 14.4-4.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for typical MSE wall stability check is presented in Table 14.6-4. Computed factored load and moments are used for performing stability checks.

14.6.3.5.2 Sliding Stability

The stability should be computed in accordance with **LRFD [11.10.5.3]** and **LRFD [10.6.3.4]**. The sliding stability analysis shall also determine the minimum resistance along the following potential surfaces in the zones shown in **LRFD [Figure 11.10.2.1]**.

- Sliding within the reinforced backfill (performed by contractor)
- Sliding along the reinforced back-fill/base soil interface (performed by designer)

The coefficient of friction angle shall be determined as:

- For discontinuous reinforcements, such as strips – the lesser of friction angle of either reinforced backfill, ϕ_r, the foundation soil, ϕ_{fd}.
- For continuous reinforcements, such as grids and sheets – the lesser of ϕ_r or ϕ_{fd} and ρ.

No passive soil pressure is allowed to resist sliding. The component of the passive resistance shall be ignored due to the possibility that permanent or temporary excavations in front of the wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance. The shear strength of the facing system is also ignored.

For adequate stability, the factored resistance should be greater than the factored load for sliding,

The following equation shall be used for computing sliding:

$R_\tau = \phi\ R_n = \phi_\tau$*(V)*(tan$\delta$)

Where:

R_R	=	Factored resistance against failure by sliding
R_n	=	Nominal sliding resistance against failure by sliding
R_τ	=	Nominal sliding resistance between soil and foundation
ϕ_τ	=	Resistance factor for shear between the soil and foundation per **LRFD [Table 11.5.6.1]**; 1.0
V	=	Factored vertical dead load
δ	=	Friction angle between foundation and soil
ρ	=	Maximum soil reinforcement interface angle **LRFD [11.11.5.3]**
tanδ	=	tan ϕ_{fd} where ϕ is lesser of (ϕ_τ , ϕ_{fd} , ρ)
H_{tot}	=	Factored total horizontal load for Strength Ia
CDR	=	$R_\tau/H_{tot} \geq 1$

14.6.3.5.3 Eccentricity Check

The eccentricity check is performed in accordance with **LRFD [11.6.3.3]** and using procedure given in publication, *FHWA-NHI-10-025*

The eccentricity is computed using:

$e = B/2 - X_0$

Where:

$$X_0 = \frac{\sum M_V - M_H}{\Sigma V}$$

Where:

ΣM_V = Summation of Resisting moment due to vertical earth pressure

ΣM_H = Summation of Moments due to Horizontal Loads

ΣV = Summation of Vertical Loads

For eccentricity to be considered acceptable, the calculated location of the resultant vertical force (based on factored loads) should be within the middle two-thirds of the base width for soil foundations (i.e., e_{max} = B/3) and middle nine-tenths of the base width for rock foundations (i.e., e_{max} = 0.45B). Therefore, for each load group, e must be less than e_{max}. If e is greater than e_{max}, a longer length of reinforcement is required. The CDR for eccentricity should be greater than 1.

$CDR = e_{max}/e > 1$

14.6.3.5.4 Bearing Resistance

The bearing resistance check shall be performed in accordance with **LRFD [11.10.5.4]**. Provisions of **LRFD [10.6.3.1]** and **LRFD [10.6.3.2]** shall apply**.** Because of the flexibility of MSE walls, an equivalent uniform base pressure shall be assumed. Effect of live load surcharge shall be added, where applicable, because it increases the load on the foundation. Vertical stress, $\sigma_{v,}$ shall be computed using following equation.

The bearing resistance computation requires:

Base Pressure $(\sigma_v) = \dfrac{\sum V}{B - 2e}$

σ_v = Vertical pressure

ΣV = Sum of all vertical forces

B = Reinforcement length

e = Eccentricity = $B/2 - X_0$

X_0 = $(\Sigma M_R - \Sigma M_H)/\Sigma V$

ΣM_V = Total resisting moments

ΣM_H = Total driving moments

The nominal bearing resistance, q_n, shall be computed using methods for spread footings. The appropriate value for the resistance factor shall be selected from **LRFD [Table 11.5.7-1]**.

The computed vertical stress, σ_v, shall be compared with factored bearing resistance, q_r in accordance with the **LRFD [11.10.5.4]** and a Capacity Demand Ratio, CDR, shall be calculated using the following equation:

$$q_r = \phi_b \, q_n \geq \sigma_v$$

Where:

q_r	=	Factored bearing resistance
q_n	=	Nominal bearing resistance computed using **LRFD [10.6.3.1.2a-1]**
ϕ_b	=	0.65 using **LRFD [Table 11.5.7-1]**
CDR	=	q_r/σ_v >1.0

14.6.3.6 Vertical and Lateral Movement

Excessive MSE wall foundation settlement can result in damage to the wall facing, coping, traffic barrier, bridge superstructure, bridge end panel, pavement, and/or other settlement-sensitive elements supported on or near the wall.

Techniques to reduce damage from post-construction settlements and deformations may include full-height vertical sliding joints through the rigid wall facing elements and appurtenances, and/or ground improvement or reinforcement techniques. Staged preload/surcharge construction using onsite materials or imported fills may also be used.

Settlement shall be computed using the procedures outlined in 14.4.7.2 and the allowable limit settlement guidelines in 14.4.7.2.1 and in accordance with **LRFD [11.10.4]** and **LRFD [10.6.2.4].** Differential settlement from the front face to the back of the wall shall be evaluated, as appropriate.

For MSE walls with rigid facing concrete panels, slip joints of 0.75 inch width can be provided to control differential settlement as per **LRFD [Table C11.104.4-1]**.

14.6.3.7 Overall Stability

Overall Stability shall be performed in accordance with **LRFD [11.10.4.3]**. Provision of **LRFD [11.6.2.3]** shall also apply. Overall and compound stability of complex MSE wall system shall also be investigated, especially where the wall is located on sloping or soft ground where overall stability may be inadequate. Compound external stability is the responsibility of the contractor/wall supplier. The long term strength of each backfill reinforcement layer intersected by the failure surface should be considered as restoring forces in the limit equilibrium slope stability analysis. Figure 14.6-4 shows failure surfaces generated during overall or compound stability evaluation.

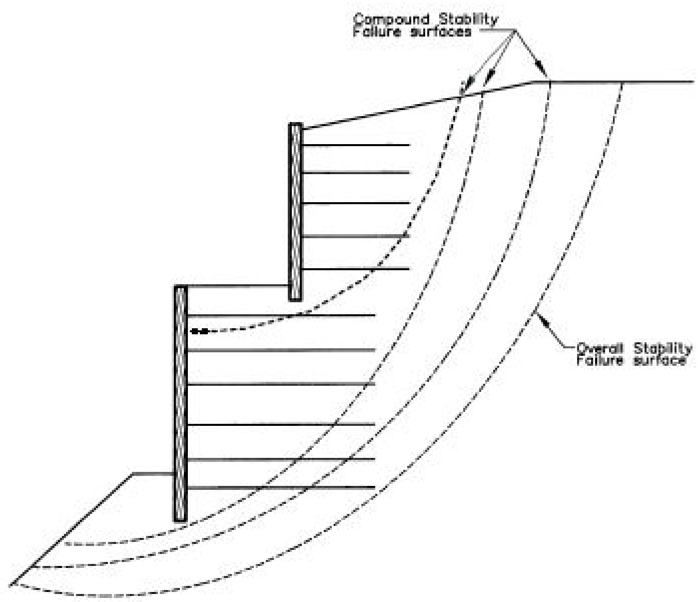


Figure 11.10.4.3-1 Overall and Compound Stability of Complex MSE Wall Systems.

Figure 14.6-4
MSE Walls Overall and Compound Stability
(Source *AASHTO LRFD*)

14.6.3.8 Internal Stability

Internal stability of MSE walls shall be performed by the wall contractor/supplier. The internal stability (safety against structural failure) shall be performed in accordance with **LRFD [11.10.6]** and shall be evaluated with respect to following at the Strength Limit:

- Tensile resistance of reinforcement to prevent breakage of reinforcement
- Pullout resistance of reinforcement to prevent failure by pullout
- Structural resistance of face elements and face elements connections

14.6.3.8.1 Loading

Figure 14.4-11 shows internal failure mechanism of MSE walls due to tensile and pullout failure of the soil reinforcement. The maximum factored tension load (T_{max}) due to tensile and pullout reinforcement shall be computed at each reinforcement level using the *Simplified Method* approach in accordance with **LRFD [11.10.6.2]**. Factored load applied to the reinforcement-facing connection (T_0) shall be equal to maximum factored tension reinforcement load (T_{max}) in accordance with **LRFD [11.10.6.2.2].**

14.6.3.8.2 Reinforcement Selection Criteria

At each reinforcement level, the reinforcement must be sized and spaced to preclude rupture under the stress it is required to carry and to prevent pullout for the soil mass. The process of sizing and designing the reinforcement consists of determining the maximum developed tension loads, their location, along a locus of maximum stress and the resistance provided by reinforcement in pullout capacity and tensile strength.

Soil reinforcements are either extensible or inextensible as discussed in 14.6.2.2.

When inextensible reinforcements are used, the soil deforms more than the reinforcement. The critical failure surface for this reinforcement type is determined by dividing the zone into active and resistant zones with a bilinear failure surface as shown in part (a) of Figure 14.6-5**.**

When extensible reinforcements are used, the reinforcement deforms more than soil and it is assumed that shear strength is fully mobilized and active earth pressure developed. The critical failure surface for both horizontal and sloping backfill conditions are represented as shown in lower part (b) of Figure 14.6-5**.**

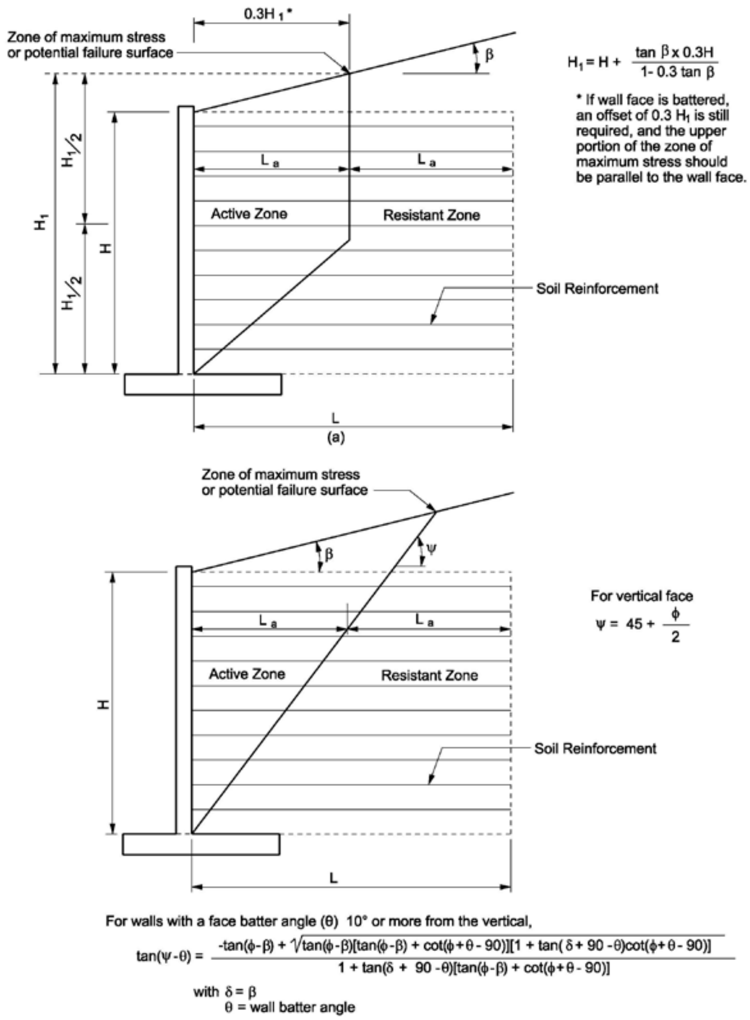


Figure 14.6-5
Location of Potential Failure Surface for Internal Stability of MSE Walls
(Source *AASHTO LRFD*)

14.6.3.8.3 Factored Horizontal Stress

The *Simplified Method* is used to compute maximum horizontal stress and is computed using the equation

$$\sigma_H = \gamma_P (\sigma_v k_r + \Delta\sigma_H)$$

Where:

γ_P = Maximum load factor for vertical stress (EV)

k_r = Lateral earth pressure coefficient computed using k_r/k_a

σ_V = Pressure due to reinforce soil mass and any surcharge loads above it

$\Delta\sigma_H$ = Horizontal stress at reinforcement level resulting in a concentrated horizontal surcharge load

Research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus extensibility, and density of reinforcement. Based on this research, a relationship between the type of reinforcement and the overburden stress has been developed and is shown in Figure 14.6-6.

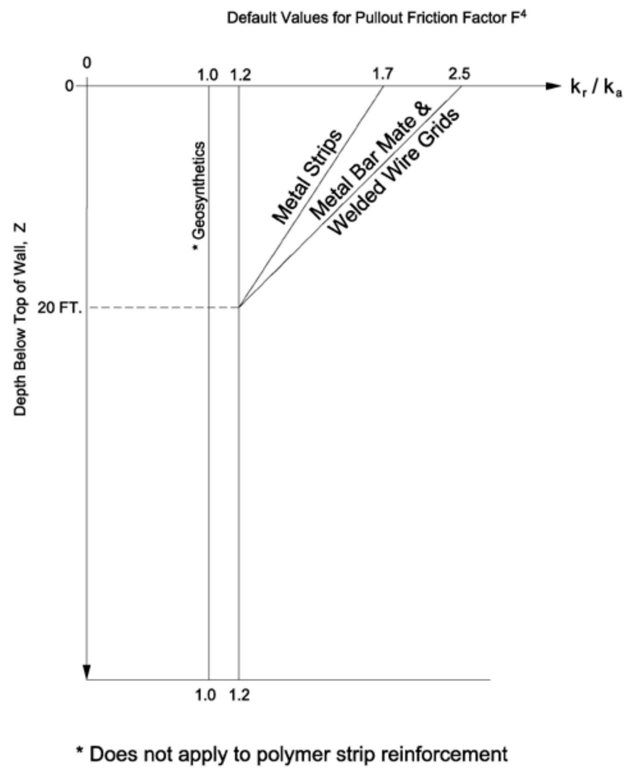


Figure 14.6-6
Variation of the Coefficient of Lateral Stress Ratio with Depth
(Source *AASHTO LRFD*)

Lateral stress ratio $k_r/k_{a,}$ can be used to compute k_r at each reinforcement level. For vertical face batter <10^0, K_a is obtained using Rankine theory. For wall face with batter greater than 10^0 degrees, Coulomb's formula is used. If present, surcharge load should be added into the estimation of $\sigma_{V.}$. For the simplified method, vertical stress for the maximum reinforcement load calculations are shown in Figure 14.6-7 .

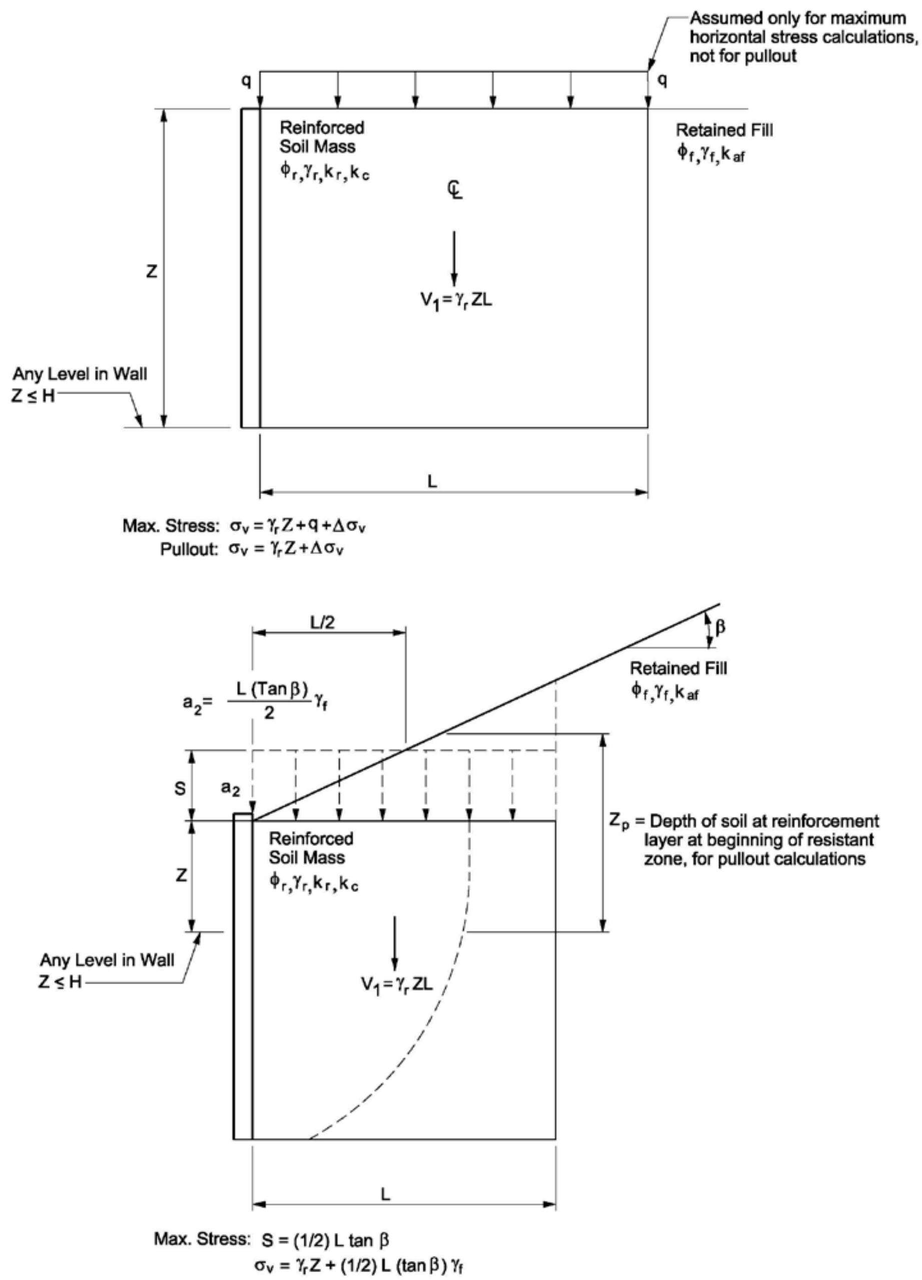


Figure 14.6-7

Calculation of Vertical Stress for Horizontal and Sloping Backslope for Internal Stability (Source *AASHTO LRFD*)

14.6.3.8.4 Maximum Factored Tension Force

The maximum tension load also referred as maximum factored tension force is applied to the reinforcements layer per unit width of wall (T_{max}) will be based on the reinforcement vertical spacing (S_V) as under:

$$T_{max} = \sigma_H S_V$$

Where:

T_{max} = Maximum tension load

σ_H = Factored horizontal load defined in 14.6.3.8.3

$T_{max\text{-}UWR}$ may also be computed at each level for discrete reinforcements (metal strips, bar mats, grids, etc) per a defined unit width of reinforcement

$T_{max\text{-}UWR}$ = $(\sigma_H S_V)/R_C$

R_C = Reinforcement coverage ratio **LRFD [11.10.6.4.1]**

14.6.3.8.5 Reinforcement Pullout Resistance

MSE wall reinforcement pullout capacity is calculated in accordance with **LRFD [11.10.6.3]**. The potential failure surface for inextensible and extensible wall system and the active and resistant zones are shown in Figure 14.6-5. The pullout resistance length, L_e, shall be determined using the following equation

$$\phi L_e = \frac{T_{max}}{\left(F^* \cdot \alpha \cdot \sigma'_v \cdot C \cdot R_c\right)}$$

Where:

L_e = Length of reinforcement in the resistance zone

T_{max} = Maximum tension load

ϕ = Resistance factor for reinforcement pullout

F* = Pullout friction factor, Figure 14.6-8

α = Scale correction factor

σ'_V = Unfactored effective vertical stress at the reinforcement level in the resistance zone

C = 2 for strip, grid, and sheet type reinforcement

R_c = Reinforcement coverage ratio **LRFD [11.10.6.4.3.2.1]**

The correction factor, α, depends primarily upon the strain softening of compacted granular material , and the extensibility, and the length of the reinforcement. Typical value is given in Table 14.6-2.

Reinforcement Type	α
All steel reinforcement	1.0
Geogrids	0.8
Geotextiles	0.6

Table 14.6-2
Typical values of α
(Source **LRFD [Table 11.10.6.3.2-1]**)

The pullout friction factor, F*, can be obtained accurately from laboratory pullout tests performed with specific material to be used on the project. Alternating, lower bound default values can be used from the laboratory or field pull out test performed in the specific back fill to be used on the project.

As shown in Figure 14.6-5, the total length of reinforcement (L) required for the internal stability is computed as below

$$L = L_e + L_a$$

Where:

L_e = Length of reinforcement in the resistance zone

L_a = Remainder length of reinforcement

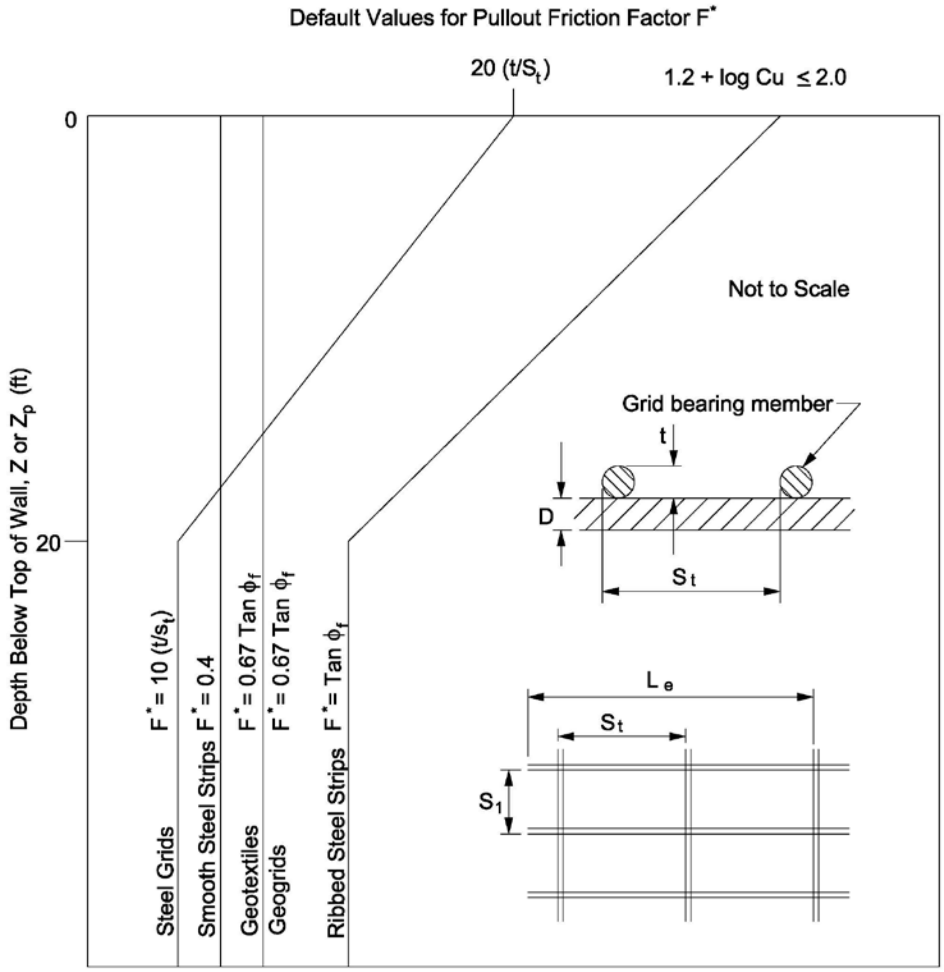


Figure 14.6-8
Typical Values of F*
(Source: **LRFD Figure [11.10.6.3.2-1]**)

14.6.3.8.6 Reinforced Design Strength

The maximum factored tensile stress (T_{MAX}) in each reinforcement layer as determined in 14.6.3.8.4 is compared to the long term reinforcement design strength computed in accordance with **LRFD [11.10.6.4.1]** as:

$$T_{MAX} \leq \phi T_{al} R_C$$

Where

ϕ = Resistance factor for tensile resistance

R_c = Reinforcement coverage ratio

T_{al} = Nominal tensile resistance (reinforcement design strength) at each reinforcement level

The value for T_{MAX} is calculated with a load factor of 1.35 for vertical earth pressure, EV. The tensile resistance factor for metallic and geosynthetic reinforcement is based on the following:

Metallic Reinforcement	Strip Reinforcement	
	• Static Loading	0.75
	Grid Reinforcement	
	• Static Loading	0.65
Geosynthetic reinforcement	• Static Loading	0.90

Table 14.6-3
Resistance Factor for Tensile and Pullout Resistance
(Source **LRFD [Table 11.5.7.1]**)

14.6.3.8.7 Calculate T_{al} for Inextensible Reinforcements

T_{al} for inextensible reinforcements is computed as below:

$T_{al} = (A_c F_y)/b$

Where:

F_y = Minimum yield strength of steel

b = Unit width of sheet grid, bar, or mat

A_c = Design cross sectional area corrected for corrosion loss

14.6.3.8.8 Calculate T_{al} for Extensible Reinforcements

The available long-term strength, T_{al}, for extensible reinforcements is computed as:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{Tult}{RF_{ID} * RF_{CR} * RF_D}$$

Where:

T_{ult} = Minimum average roll value ultimate tensile strength

RF = Combined strength reduction factor to account for potential long term degradation due to installation, damage, creep, and chemical aging

RF_{ID} = Strength Reduction Factor related to installation damage

RF_{CR} = Strength Reduction Factor caused by creep due to long term tensile load

RF_D = Strength Reduction Factor due to chemical and biological degradation

RF shall be determined from product specific results as specified in **LRFD [11.10.6.4.3b]**.

14.6.3.8.9 Design Life of Reinforcements

Long term durability of the steel and geosynthetic reinforcement shall be considered in MSE wall design to ensure suitable performance throughout the design life of the structure.

The steel reinforcement shall be designed to achieve a minimum designed life in accordance with **LRFD [11.5.1]** and shall follow the provision of **LRFD [7.6.4.2]**. The provision for corrosion loss shall be considered in accordance with the guidance presented in **LRFD [11.10.6.4.2a]**.

The durability of polymeric reinforcement is influenced by time, temperature, mechanical damage, stress levels, and changes in molecular structure. The strength reduction for geosynthetic reinforcement shall be considered in accordance with **LRFD [11.10.6.4.2b]**.

14.6.3.8.10 Reinforcement /Facing Connection Design Strength

Connections shall be designed to resist stresses resulting from active forces as well as from differential movement between the reinforced backfill and the wall facing elements in accordance with **LRFD [11.10.6.4.4]**.

<u>Steel Reinforcement</u>

Capacity of the connection shall be tested per **LRFD [5.11.3].** Elements of the connection which are embedded in facing elements shall be designed with adequate bond length and bearing area in the concrete, to resist the connection forces. The steel reinforcement connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4a]**.

Connections between steel reinforcement and the wall facing units (e.g. bolts and pins) shall be designed in accordance with **LRFD [6.1.3]**. Connection material shall also be designed to accommodate loss due to corrosion.

<u>Geosynthetic Reinforcement</u>

The portion of the connection embedded in the concrete facing shall be designed in accordance with **LRFD [5.11.3]**. The nominal geosynthetic connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4b]**.

14.6.3.8.11 Design of Facing Elements

Precast Concrete Panel facing elements are designed to resist the horizontal forces developed internally within the wall. Reinforcement is provided to resist the average loading conditions at each depth in accordance with structural design requirements in *AASHTO LRFD*. The embedment of the reinforcement to panel connector must be developed by test, to ensure that it can resist the maximum tension. The concrete panel must meet temperature and shrinkage steel requirements. Epoxy protection of panel reinforcement is required.

Modular Block Facing elements must be designed to have sufficient inter-unit shear capacity. The maximum spacing between unit reinforcement should be limited to twice the front block width or 2.7 feet, whichever is less. The maximum depth of facing below the bottom reinforcement layer should be limited to the block width of modular facing unit. The top row of reinforcement should be limited to 1.5 times the block width. The factored inter-unit shear capacity as obtained by testing at the appropriate normal load should exceed the factored horizontal earth pressure.

14.6.3.8.12 Corrosion

Corrosion protection is required for all permanent and temporary walls in aggressive environments as defined in **LRFD [11.10.2.3.3].** Aggressive environments in Wisconsin are typically associated with salt spray and areas near storm water pipes in urban areas. MSE walls with steel reinforcement should be protected with a properly designed impervious membrane layer below the pavement and above the first level of the backfill reinforcement. The details of the impervious layer drainage collector pipe can be found in *FHWA-NHI-0043* (FHWA 2001).

14.6.3.9 Wall Internal Drainage

The wall internal drainage should be designed using the guidelines provided in 14.4.7.6. Pipe underdrain must be provided to properly drain MSE walls. Chimney or blanket drains with collector-pipe drains are installed as part of the MSE walls sub-drainage system. Collector pipes with solid pipes are required to carry the discharge away from the wall. All collector pipes and solid pipes should be 6-inch diameter.

14.6.3.10 Traffic Barrier

Design concrete traffic barriers on MSE walls to distribute applied traffic loads in accordance with **LRFD [11.10.10.2]** and WisDOT standard details. Traffic impact loads shall not be transmitted to the MSE wall facing. Additionally, MSE walls shall be isolated from the traffic barrier load. Traffic barrier shall be self-supporting and not rely on the wall facing.

14.6.3.11 Design Example

Example E-2 shows a segmental precast panel MSE wall with steel reinforcement. Example E-3 shows a segmental precast panel MSE wall with geogrid reinforcement. Both design examples include external and internal stability of the walls. The design examples are included in 14.18.

14.6.3.12 Summary of Design Requirements

1. Strength Limit Checks

   a. External Stability

   - Sliding

$$CDR = \left(\frac{R_\tau}{H_{tot}}\right) > 1.0$$

   - Eccentricity Check

$$CDR = \left(\frac{e_{\max}}{e}\right) > 1.0$$

   - Bearing Resistance

$$CDR = \left(\frac{q_r}{\sigma_v}\right) > 1.0$$

   b. Internal stability

   - Tensile Resistance of Reinforcement
   - Pullout Resistance of Reinforcement
   - Structural resistance of face elements and face elements connections

   c. Service Limit Checks

   - Overall Stability
   - Wall Settlement and Lateral Deformation

2. Concrete Panel Facings

   - f'_c = 4000 psi (wet cast concrete)
   - Min. thickness = 5.5 inches
   - Min. reinforcement = 1/8 square inch per foot in each direction (uncoated)

- Min. concrete cover = 1.5 inches
- fy = 60,000 psi

3. Traffic/ Surcharge

- Traffic live load surcharge = 240 lb/ft^2 or
- Non traffic live load surcharge =100 lb/ft^2

4. Reinforced Earthfill

- Unit weight = 120 lb/ft^3
- Angle of internal friction = 30^0, or as determined from Geotechnical analyses (maximum allowed is 36^0)

5. Retained Soil

- Unit weight = 120 lb/ft^3
- Angle of internal friction = 30^0, or as determined from Geotechnical analyses

6. Design Life

- 75 year minimum for permanent walls

7. Soil Pressure Theory

- Coulomb's Theory

8. Soil Reinforcement

For steel or geogrid systems, the minimum soil reinforcement length shall be 70 percent of the wall height and not less than 8 feet. The length of soil reinforcement shall be equal from top to bottom. Soil reinforcement must extend a minimum of 3 feet beyond the failure plane.

9. Summary of Load Combinations and Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50		Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50		Bearing, Wall strength
Extreme IIa	0.90	1.00	-	-	1.00	1.00	Sliding, eccentricity
Extreme IIb	1.25	1.35	-	-	1.00-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00	-	Global, settlement, wall crack control

Table 14.6-4
Load Factor Summary for MSE-External Stability
(Source **LRFD [Table 3.4.1]**)

10. Resistance Factors for External Stability

Stability mode	Condition	Resistance Factor
Sliding		1.00
Bearing		0.65
Overall stability	Geotechnical parameters are well defined and slope does not support a structural element	0.75
	Geotechnical parameters are based on limited information, or the slope supports a structural element	0.65

Table 14.6-5
Resistance Factor Summary for MSE-External Stability
(Source **LRFD [Table 11.5.6.1]**)

14.7 Modular Block Gravity Walls

The proprietary modular blocks used in combination with soil reinforcement "Mechanically Stabilized Earth Retaining Walls with Modular Block Facings" can also be used as pure gravity walls (no soil reinforcement). These walls consist of a single row of dry stacked blocks (without mortar) to resist external pressures. These walls can be formed to a tight radius of curvature of 50 ft. or greater. A drawback is that these walls are settlement sensitive. This wall type should only be considered when adequate provisions are taken to keep the surface water runoff and the ground water seepage away from the wall face.

The material specifications for the blocks used for gravity walls are identical to those for the blocks used for block MSE walls as discussed in 14.6.2.3. The modular block gravity walls are proprietary. The wall supplier is responsible for the design of these walls. Design drawings and calculations must be submitted to WisDOT for approval.

The height to which they can be constructed, is a function of the depth of the blocks, the setback of the blocks, the front slope and backslope angle, the surcharge on the retained soil and the angles of internal friction of the retained soil behind the wall. Walls of this type are limited to a height from top of leveling pad to top of wall of 8 feet or less, and are limited to a maximum differential settlement of 1/200.

Footings for modular block gravity walls are either base aggregate dense 1-¼ inch (Section 305 of the *Standard Specifications)* or Grade A concrete. Minimum footing thickness is 12 inches for aggregate and 6 inches for concrete. The width of the footing equals the width of the bottom block plus 12 inches for aggregate footings and plus 6 inches for concrete footings. The bottom modular block is central on the leveling pad. The standard special provisions for Modular Block Gravity Walls require a concrete footing if any portion of a wall is over 5 feet measured from the top of the footing to the bottom of the wall cap.

The coarse aggregate No. 1 (501.2.5.4 of the *Standard Specifications*), is placed within 1 foot behind the back face of the wall, extending down to the bottom of the footing.

14.7.1 Design Procedure for Modular Block Gravity Walls

All modular block gravity walls shall be designed to resist external pressure caused by the supported earth, surcharge loads, and water in accordance with the design criteria discussed in **LRFD [11.11.4]** and 14.4. The design requires an external stability evaluation including sliding, eccentricity check, and bearing resistance check at the Strength I limit state and the evaluation of wall settlement and overall stability at the Service I limit state.

The design of modular block gravity walls provided by the contractor must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in 14.15.2 and 14.16. The design must include an analysis of external stability including sliding, eccentricity, and bearing stress check. Horizontal shear capacity between blocks must also be verified by the contractor.

Settlement and overall stability calculations are the responsibility of the designer. The soil design parameters and allowable bearing capacity for the design are provided by the Geotechnical Engineer, including the minimum required block depth.

14.7.1.1 Initial Sizing and Wall Embedment

The minimum embedment to the top of the footing for modular block gravity walls is the same as stated in **LRFD [11.10.2.2]** for mechanically stabilized earth walls. Wall backfill slope shall not be steeper than 2:1. Where practical, a minimum 4.0 ft wide horizontal bench shall be provided in front of the walls.

Wall embedment for prefabricated modular walls shall meet the requirements discussed in section14.4.7.5. The minimum embedment shall be 1.5 ft. or the requirement of scouring or erosion due to flooding defined in 14.6.3.4.1.

14.7.1.2 External Stability

The external stability analyses shall develop the unfactored and factored loads and include evaluations for sliding, eccentricity check, and bearing resistance in accordance with **LRFD [11.11.4]. LRFD [11.11.4.1]** requires that wall stability be performed at every block level.

14.7.1.2.1 Unfactored and Factored Loads

Unfactored loads and moments shall be computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure 14.4-5 shall be developed. Factored loads and moments shall be computed as discussed in 14.4.6 by multiplying applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for a typical modular block wall is presented in Table 14.7-1. Computed factored load and moments are used for performing stability checks.

14.7.1.2.2 Sliding Stability

Sliding should be considered for the full height wall and at each block level in the wall. The stability should be computed in accordance with **LRFD [10.6.3.4]**, using the following equation:

$$R_R = \phi\, R_n = \phi_\tau\, R_\tau$$

Where:

R_R	=	Factored resistance against failure by sliding
R_n	=	Nominal sliding resistance against failure by sliding
ϕ_τ	=	Resistance factor for shear between soil and foundation per **LRFD [Table 10.5.5.2.2.1]**
ϕ_τ	=	0.9 for concrete on sand and 1.0 for soil on soil
R_τ	=	Nominal sliding resistance between soil and foundation

No passive soil pressure is allowed to resist sliding. The component of the passive resistance shall be ignored due to the possibility that permanent or temporary excavations in front of the wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance.

Interface sliding resistance between concrete blocks shall be calculated using the corrected wall weight based on the calculated hinge height in accordance with **LRFD Figure [11.10.6.4.4b-1]**. Interface friction resistance parameters shall be based on NCMA method. Shear between the blocks must be resisted by friction, keys or pins.

14.7.1.2.3 Bearing Resistance

The bearing resistance of the walls shall be computed in accordance with **LRFD [10.6.3.1]**.

Base Pressure, $\sigma_v = \dfrac{\sum V_{tot}}{(B - 2e)}$

The computed vertical stress shall be compared with factored bearing resistance in accordance with the **LRFD [10.6.3.1]**, using following equation:

$q_r = \phi_b q_n \geq \sigma_v$

Where:

q_n	=	Nominal bearing resistance computed using **LRFD [10.6.3.1.2-a]**
$\sum V$	=	Summation of Vertical loads
B	=	Base width
e	=	Eccentricity
ϕ_b	=	0.55 **LRFD [Table 11.5.7-1]**

14.7.1.2.4 Eccentricity Check

The eccentricity check shall be performed in accordance with **LRFD [11.6.3.3].** The location of the resultant force should be within the middle two-thirds of the base width (e<B/3) for footings on soil, and within nine-tenths of the base (e<0.45B) for footings on rock.

14.7.1.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I limit states using procedures described in14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.7.1.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3, with the exception that the entire mass of the modular walls (or the "foundation load"), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Engineering Unit or Consultant of record.

14.7.1.5 Summary of Design Requirements

1. Stability Evaluations
   - External Stability
     - Eccentricity Check
     - Bearing Check
     - Sliding
   - Settlement
   - Overall/Global
2. Block Data
   - One piece block
   - Minimum thickness of front face = 4 inches
   - Minimum thickness of internal cavity walls other than front face = 2 inches
   - 28 day concrete strength = 5000 psi
   - Maximum water absorption rate by weight = 5%
3. Traffic Surcharge
   - Traffic live load surcharge = 240 lb/ft^2
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment
4. Retained Soil
   - Unit weight γ_f = 120 lb/ft^3
   - Angle of internal friction as determined by Geotechnical Engineer

5. Soil Pressure Theory

   - Use Coulomb Theory

6. Maximum Height = 8 ft.

   (This height is measured from top of leveling pad to bottom of cap. It is not the exposed height). In addition this maximum height may be reduced if there is sloping backfill or a sloping surface in front of the wall.)

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50		Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50		Bearing /wall strength
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

Table 14.7-1
Load Factor Summary for Prefabricated Modular Walls

8. Sliding Resistance Factors

   ϕ_τ = 1.0 **LRFD [Table11.5.7-1]**

9. Bearing Resistance Factors

   ϕ_b = 0.55 **LRFD [Table11.5.7-1]**

14.8 Prefabricated Modular Walls

Prefabricated modular walls systems use interconnected structural elements, which use selected in-fill soil or rock fill to resist external pressures by acting as gravity retaining walls. Metal and precast concrete or metal bin walls, crib walls, and gabion walls are considered under the category of prefabricated modular walls. These walls consist of modular elements which are proprietary. The design of these wall systems is provided by the contractor/wall supplier.

Prefabricated modular walls can be used where reinforced concrete walls are considered. Steel modular systems should not be used where aggressive environmental condition including the use of deicing salts or other similar chemicals are used that may corrode steel members and shorten the life of modular wall systems.

14.8.1 Metal and Precast Bin Walls

Metal bin walls generally consist of sturdy, lightweight, modular steel members called as stringers and spacers. The stringers constitute the front and back face of the bin and spacers its sides. The wall is erected by bolting the steel members together. The flexibility of the steel structure allows the wall to flex against minor ground movement. Metal bin walls are subject to corrosion damage from exposure to water, seepage and deicing salts. To improve the service life of metal bin walls, consideration should be given towards increasing the galvanizing requirements and establishing electrochemical requirements for the confined backfill.

Precast concrete bin walls are typically rectangular interlocking prefabricated concrete modules. A common concrete module typically has a face height varying from 4 to 5 feet, a face length up to 8 feet, and a width ranging from 4 to 20 feet. The wall can be assembled vertically or provided with a batter. A variety of surface treatment can be provided to meet aesthetic requirements. A parapet wall can be provided at the top of the wall and held rigidly by a cast in place concrete slab. A reinforced cast-in-place or precast concrete footing is usually placed at the toe and heel of the wall.

Bin walls are not recommended for applications that require a radius of curvature less than 800 ft. The wall face batter shall not be steeper than 10° or 6:1 (V:H). The base width of bin walls is generally 60% of the wall height. Further description and method of construction can be found in FHWA's publication *Earth Retaining Structures* 2008.

14.8.2 Crib Walls

Crib walls are built using prefabricated units which are stacked and interlocked and filled with free draining material. Cribs consist of solid interlocking reinforced concrete members called rails and tiebacks (sometimes called stretchers and headers). The rails run parallel with the wall face at both the front and rear of the cribbing and the tiebacks run transverse to the rails to tie the structure together. Rails and cross sections of tiebacks form the front face of the wall.

The wall face can either be opened or closed. In closed faced cribs, stretchers are placed in contact with each other. In open face cribs, the stretchers are placed at an interval such that the infill material does not escape through the face. The wall face batter for crib walls shall be no steeper than 4:1.

14.8.3 Gabion Walls

The gabion walls are composed of orthogonal wire cages or baskets tied together and filled with rock fragments. These wire baskets are also known as gabion baskets. The basket size can be varied to suit the terrain with a standard width of 3 feet to standard length varying 3 to 12 feet. The standard height of these baskets may vary from 1 foot to 3 feet. Individual wire baskets are filled with rock fragments ranging in size from 4 to 10 inches. After the baskets are filled, the lids are closed and wired shut to form a relatively rigid block. Succeeding rows of the gabions are laced in the field to the underlying gabions and are filled in the same manner until the wall reaches its design height. The rock filled baskets are closed with lids.

The durability of a gabion wall is dependent upon maintaining the integrity of the gabion baskets. Galvanized steel wire is required for all gabion installations. Although gabions are manufactured from a heavy gage wire, there is a potential for damage due to vandalism. While no known case of such vandalism has occurred on any existing WisDOT gabion walls, the potential for such action should be considered at specific sites.

A height of about 18 feet should be considered as a practical limit for gabion walls. Gabion walls have shown good economy for low to moderate heights but lose this economy as height increases. The front and rear face of the wall may be vertical or stepped. A batter is provided for walls exceeding heights of 10 feet, to improve stability. The wall face step shall not be steeper than 6” or 10:1(V:H). The minimum embedment for gabion walls is 1.5 feet. The ratio of the base width to height will normally range from 0.5 to 0.75 depending on backslope, surcharge and angle of internal friction of retained soil. Gabion walls should be designed in cross section with a horizontal base and a setback of 4 to 6 inches at each basket layer. This setback is an aid to construction and presents a more pleasing appearance. The use of a tipped wall base should not be allowed except in special circumstances.

14.8.4 Design Procedure

All prefabricated modular wall systems shall be designed to resist external pressure caused by the supported earth, surcharge loads, and water in accordance with design criteria discussed in **LRFD [11.11.4]** and 14.4 of this chapter. The design requires an external stability evaluation by the WISDOT/Consultant designer, including sliding, eccentricity, and bearing resistance check at the Strength I limit state and the evaluation of wall settlement and overall stability at the Service I limit state.

In addition, the structures modules of the bin and crib walls shall be designed to provide adequate resistance against structural failure as part of the internal stability evaluations in accordance with the guidelines presented in **LRFD [11.11.5]**.

No separate guidance is provided in the *AASHTO LRFD* for the gabion walls, therefore, gabion walls shall be evaluated for the external stability at Strength I and the settlement and overall stability checks at Service I using similar process as that of a prefabricated modular walls.

Since structure modules of the prefabricated modular walls are proprietary, the contractor/ supplier is responsible for the internal stability evaluation and the structural design of the structural modules. The design by contractor shall also meet the requirements for any special provisions. The external stability, overall stability check and the settlement evaluation will be performed by Geotechnical Engineer.

14.8.4.1 Initial Sizing and Wall Embedment

Wall backfill shall not be steeper than 2:1(V:H). Where practical, a minimum 4.0 feet wide horizontal bench shall be provided in front of the walls. A base width of 0.4 to 0.5 of the wall height can be considered initially for walls with no surcharge. For walls with surcharge loads or larger backslopes, an initial base width of 0.6 to 0.7 times can be considered.

Wall embedment for prefabricated modular walls shall meet the requirements discussed in 14.4.7.5. A minimum embedment shall be 1.5 ft or the requirement for scouring or erosion due to flooding.

14.8.5 Stability checks

Stability computations for crib, bin, and gabion modular wall systems shall be made by assuming that the wall modules and wall acts as a rigid body. Stability of gabion walls shall be performed assuming that gabions are flexible.

14.8.5.1 Unfactored and Factored Loads

All modular walls shall be investigated for lateral earth and water pressure including any live and/or dead load surcharge. Dead load due to self-weight and soil or rock in-fill shall also be included in computing the unfactored loads. Material properties for selected backfill, concrete, and steel shall be in accordance with guidelines suggested in 14.4.6. The properties of prefabricated modules shall be based on the type of wall modules being supplied by the wall suppliers.

The angle of friction δ between the back of the modules and backfill shall be used in accordance with the **LRFD [3.11.5.9]** and **LRFD [Table C3.11.5.9.1]**. Loading and earth pressure distribution diagram shall be developed as shown in Figure 14.4-6 or Figure 14.4-7

Since infill material and backfill materials of the gabion walls are well drained, no hydrostatic pressure is considered for the gabion walls. The unit weight of the rock-filled gabion baskets shall be computed in accordance with following:

$$\gamma_g = (1-\eta_r)G_s\gamma_w$$

Where:

η_r	=	Porosity of the rock fill
Gs	=	Specific gravity of the rock

γ_w = Unit weight of water

Free-draining granular material shall be used as backfill material behind the prefabricated modules in a zone of 1:1 from the heel of the wall. The soil design parameters shall be provided by the Geotechnical Engineer.

Factored loads and moments shall be computed as discussed in 14.4.5.5 and shall be multiplied by applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for a typical modular block wall is presented in Table 14.8-1

14.8.5.2 External Stability

The external stability of the prefabricated modular walls shall be evaluated for sliding, eccentricity check, and bearing resistance in accordance with **LRFD [11.11.4].** It is assumed that the wall acts as a rigid body. **LRFD [11.11.4.1]** requires that wall stability be performed at every module level. The stability can be evaluated using procedure described in 14.7.1.2.

For prefabricated modular walls, the sliding analysis shall be performed by assuming that 80% of the weight of the soil in the modules is transferred to the footing supports with the remaining soil, weight being transferred to the area of the wall between footings.

The load resisting overturning shall also be limited to 80%, because the interior of soil can move with respect to the retaining module.

The bearing resistance shall be evaluated by assuming that 80% weight of the infill soil is transferred to point (or line) supports at the front or rear of the module.

14.8.5.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I using procedure described in 14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.8.5.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3 with the exception that the entire mass of the modular walls (or the "foundation load"), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Engineer.

14.8.5.5 Structural Resistance

Structural design of the modular units or members shall be performed in accordance with **LRFD [11.11.5]**. The design shall be performed using the factored loads developed for the geotechnical design (external stability) and for the factored pressures developed inside the modules in accordance with **LRFD [11.11.5.1]**. Design shall consider any potential failure mode, including tension, compression, shear, bending, and torsion. The contractor/wall supplier is responsible for the structural design of wall components.

14.8.6 Summary of Design Safety Factors and Requirements

Requirements

Stability Checks

- External Stability
    - Sliding
    - Overturning (eccentricity check)
    - Bearing Stress
- Internal Stability
    - Structural Components
- Settlement
- Overall Stability

Foundation Design Parameters

- Use values provided by Geotechnical Engineer

Concrete and steel Design Data

- f'_c = 4000 psi (or as required by design)
- f_y = 60,000 psi

Use uncoated bars or welded wire fabric

Traffic Surcharge

- Traffic live load surcharge = 240 lb/ft^2
- If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment

Retained Soil

- Unit weight = 120 lb/ft^3
- Angle of internal friction =
  - Use value provided by Geotechnical Engineer
- Rock-infill unit weight =
  - Based on porosity and rock type

Soil Pressure Theory

- Coulomb's Theory for prefabricated wall systems
- Rankine theory or Coulomb theory, at the discretion of designer for gabion walls

7 Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{ES}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50	1.50	Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	1.50	Bearing /wall strength
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

Table 14.8-1
Load Factor Summary for Prefabricated Modular Walls

14.9 Soil Nail Walls

Soil nail walls consist of installing reinforcement of the ground behind an excavation face, by drilling and installing closely-spaced rows of grouted steel bars (i.e., soil nails). The soil nails are grouted in place and subsequently covered with a facing; used to stabilize the exposed excavation face, support the sub-drainage system (i.e., composite strip drain, collector and drainage pipes), and distribute the soil nail bearing plate load over a larger area. When used for permanent applications, a permanent facing layer, meeting the aesthetic and structural requirement is constructed directly over the temporary facing.

Soil nail walls are typically used to stabilize excavation during construction. Soil nail walls have been used recently with MSE walls to form hybrid wall systems typically known as 'shored walls'. The soil nails are installed as top down construction. Conventional soil nail wall systems are best suited for sites with dense to very dense, granular soil with some apparent cohesion (sands and gravels), stiff to hard, fine-grained soil (silts and clays) of relatively low plasticity (PI<15), or weak, weathered massive rock with no adversely-oriented planes of weakness. Soil nail wall construction requires that open excavations stand unsupported long enough to allow soil nail drilling and grouting, sub-drainage installation, reinforcement, and temporary shotcrete placement. Soil nail walls should not be used below groundwater.

14.9.1 Design Requirements

AASHTO LRFD currently does not include the design and construction of soil nail walls. It is recommended that soil nail walls be designed using methods recommended in *Geotechnical Engineering Circular (GEC) No. 7 – Soil Nail Walls* (FHWA, 2003). The design life of the soil nail walls shall be in accordance with 14.4.3.

The design of the soil nailing walls requires an evaluation of external, internal, and overall stability and facing-connection failure mode as presented in Sections 5.1 thru Sections 5.6 of *(GEC) No. 7 – Soil Nail Walls* (FHWA, 2003).

A permanent wall facing is required for all permanent soil nail walls. Permanent facing is commonly constructed of cast-in-place (CIP) concrete, welded wire mesh (WWM) reinforced concrete and precast fabricated panels. In addition to meeting the aesthetic requirements and providing adequate corrosion protections to the soil nails, design facings for all facing-connection failure modes indicated in FHWA 2003.

Corrosion protection is required for all permanent soil nail wall systems to assure adequate long-term wall durability. . The level of corrosion protection required should be determined on a project-specific basis based on factors such as wall design life, structure criticality and the electrochemical properties of the supporting soil and rock materials. Criteria for classification of the supporting soil and rock materials as "aggressive" or "non-aggressive" are provided in FHWA 2003.

Soil nails are field tested to verify that nail design loads can be supported without excessive movement and with an adequate margin of safety. Perform both verification and proof testing of designated test nails as recommended in FHWA 2003.

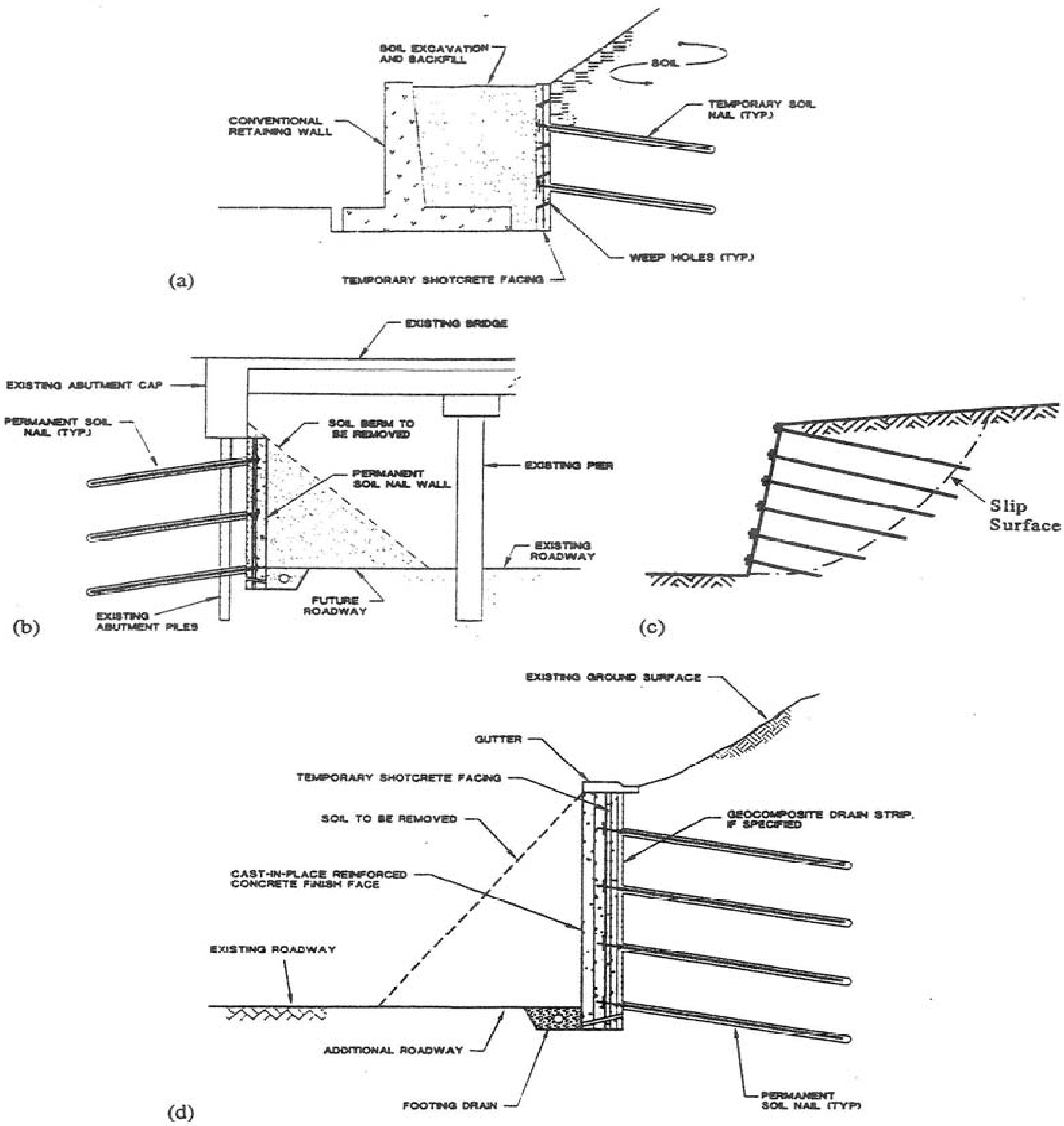


Figure 14.9-1
In-Situ Soil Nailed Walls
(Source: Earth Retaining Structures, 2008)

14.10 Steel Sheet Pile Walls

14.10.1 General

Steel sheet pile walls are a type of non-gravity wall and are typically used as temporary walls, but can also be used for permanent locations.

Sheet piling consists of interlocking steel, precast concrete or wood pile sections driven side by side to form a continuous unit. Steel is used almost exclusively for sheet pile walls. Individual pile sections usually vary from 12 to 21 inches in width, allowing for flexibility and ease of installation. The most common use of sheet piling is for temporary construction of cofferdams, retaining walls or trench shoring. The structural function of sheet piles is to resist lateral pressures due to earth and/or water. The steel manufacturers have excellent design references. Sheet pile walls generally derive their stability from sufficient pile penetration (cantilever walls). When sheet pile walls reach heights in excess of approximately 15 feet, the lateral forces are such that the walls need to be anchored with some form of tieback.

Cofferdams depend on pile penetration, ring action and the tensile strength of the interlocking piles for stability. If a sheet pile cofferdam is to be dewatered, the sheets must extend to a sufficient depth into firm material to prevent a "blow out", that is water coming in from below the base of the excavation. Cross and other bracing rings must be adequate and placed as quickly as excavation permits.

Sheet piling is generally chosen for its efficiency, versatility, and economy. Cofferdam sheet piling and any internal bracing are designed by the Contractor, with the design being accepted by the Department. Other forms of temporary sheet piling are designed by the Department. Temporary sheet piling is not the same as temporary shoring. Temporary shoring is designed by the Contractor and may involve sheet piling or other forms of excavation support.

14.10.2 Sheet Piling Materials

Although sheet piling can be composed of timber or precast concrete members, these material types are seldom, if ever, used on Wisconsin transportation projects.

Steel sheet piles are by far the most extensively used type of sheeting in temporary construction because of their availability, various sizes, versatility and ability to be reused. Also, they are very adaptable to permanent structures such as bulkheads, seawalls and wharves if properly protected from salt water.

Sheet pile shapes are generally Z, arched or straight webbed. The Z and the medium to high arched sections have high section moduli and can be used for substantial cantilever lengths or relatively high lateral pressures. The shallow arched and straight web sections have high interlocking strength and are employed for cellular cofferdams. The Z-section has a ball-and-socket interlock and the arched and straight webbed sections have a thumb-and-finger interlock capable of swinging 10 degrees. The thumb-and-finger interlock provides high tensile strength and considerable contact surface to prevent water passage. Continuous steel sheet piling is not completely waterproof, but does stop most water from passing through the joints. Steel sheet piling is usually 3/8 to 1/2 inch thick. Designers should specify the required section modulus and embedment depths on the plans, based on bending requirements and also account for corrosion resistance as appropriate.

Refer to steel catalogs for typical sheet pile sections. Contractors are allowed to choose either hot or cold rolled steel sections meeting the specifications. Previously used steel sheet piling may be adequate for some temporary situations, but should not be allowed on permanent applications.

14.10.3 Driving of Sheet Piling

All sheets in a section are generally driven partially to depth before all are driven to the final required depths. There is a tendency for sheet piles to lean in the direction of driving producing a net "gain" over their nominal width. Most of this "gain" can be eliminated if the piles are driven a short distance at a time, say from 6 feet to one third of their length before any single pile is driven to its full length. During driving if some sheet piles strike an obstruction, move to the next pile that can be driven and then return to the piles that resisted driving. With interlock guides on both sides and a heavier hammer, it may be possible to drive the obstructed sheet to the desired depth.

Sheet piles are installed by driving with gravity, steam, air or diesel powered hammers, or by vibration, jacking or jetting depending on the subsurface conditions, and pile type. A vibratory or double acting hammer of moderate size is best for driving sheet piles. For final driving of long heavy piles a single acting hammer may be more effective. A rapid succession of blows is generally more effective when driving in sand and gravel; slower, heavier blows are better for penetrating clay materials. For efficiency and impact distribution, where possible, two sheets are driven together. If sheets adjacent to those being driven tend to move down below the required depth, they are stopped by welding or bolting to the guide wales. When sheet piles are pulled down deeper than necessary by the driving of adjacent piles, it is generally better to fill in with a short length at the top, rather than trying to pull the sheet back up to plan location.

14.10.4 Pulling of Sheet Piling

Vibratory hammers are most effective in removing sheets and typically used. Sheet piles are pulled with air or steam powered extractors or inverted double acting hammers rigged for this application. If piles are difficult to pull, slight driving is effective in breaking them loose. Pulled sheet piling is to be handled carefully since they may be used again; perhaps several times.

14.10.5 Design Procedure for Sheet Piling Walls

A description of sheet pile design is given in **LRFD [11.8.2]** as **"**Cantilevered Wall Design" along with the earth pressure diagrams showing some simplified earth pressures. They are also referred to as flexible cantilevered walls. Steel sheet pile walls can be designed as cantilevered walls up to approximately 15 feet in height. Over 15 feet height, steel sheet pile walls may require tie-backs with either prestressed soil anchors, screw anchors, or deadman-type anchors.

The preferred method of designing cantilever sheet piling is by the "Conventional Method" as described in the *United States Steel Sheet Piling Design Manual* (February,1974). The Geotechnical Engineer provides the soil design parameters including cohesion values, angles of internal friction, wall friction angles, soil densities, and water table elevations. The lateral earth pressures for non-gravity cantilevered walls are presented in **LRFD [3.11.5.6].**

Anchored wall design must be in accordance with **LRFD [11.5.6]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

All areas of permanent exposed steel sheet piling above the ground line shall be coated or painted prior to driving. Corrosion potential should be considered in all steel sheet piling designs. Special consideration should be given to permanent steel sheet piling used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see *Facilities Development Manual*, Procedure 13-1-15).

Permanent sheet pile walls below the watertable may require the use of composite strip drains, collector and drainage pipes before placement of the final concrete facing.

The appearance of permanent steel sheet piling walls may be enhanced by applying either precast concrete panels or cast-in-place concrete surfacing. Welded stud-shear connectors can be used to attach cast-in-place concrete to the sheet piling. Special surface finishes obtained by using form liners or other means and concrete stain or a combination of stain and paint can be used to enhance the concrete facing aesthetics.

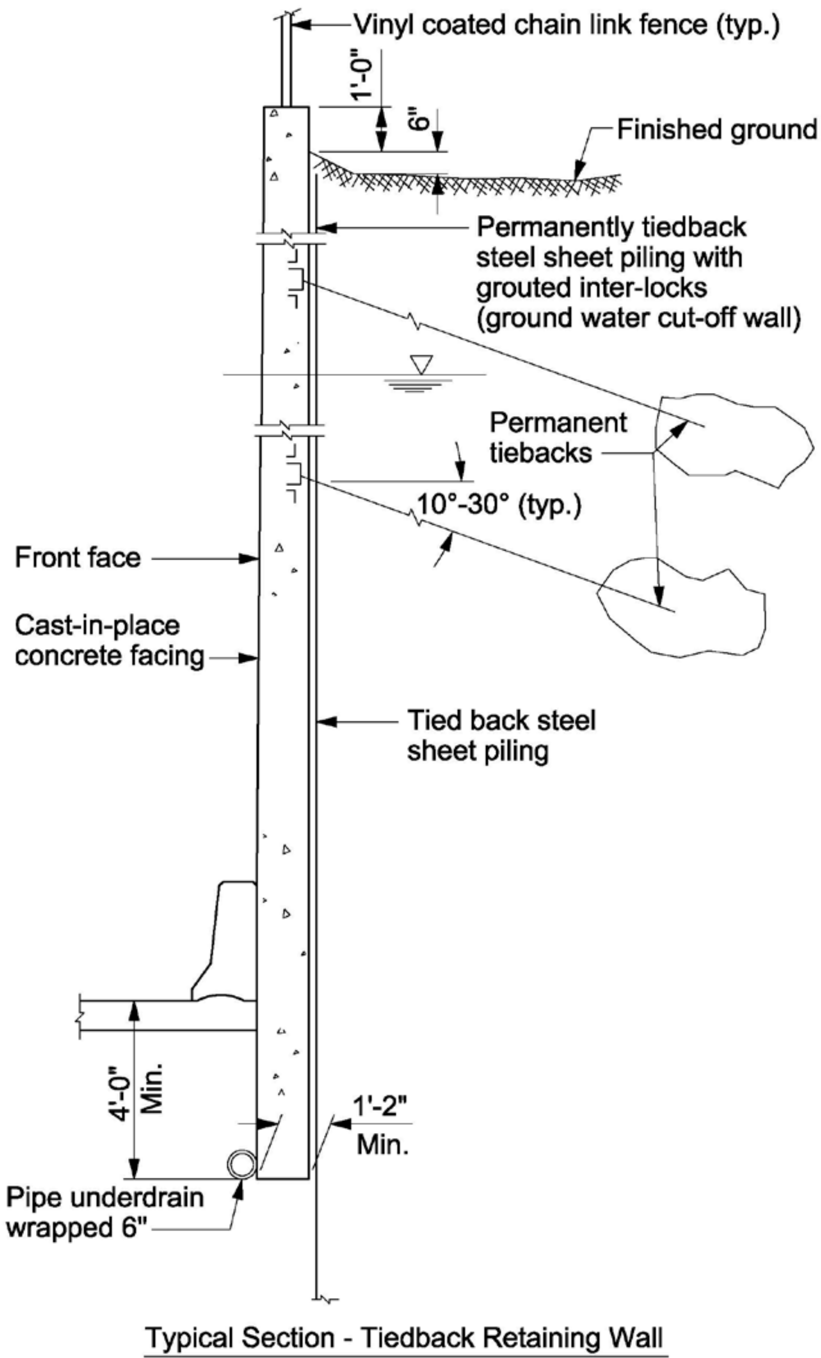


Figure 14.10-1
Typical Anchored Sheet Pile Wall

14.10.6 Summary of Design Requirements

1. Load and Resistance Factor

Load Combination	Load Factors	Resistance Factor
Strength I (maximum)	EH-Horizontal Earth Pressure: δ =1.50 **LRFD [Table 3.4.1-2]**	-----------
Strength I (maximum)	LS-Live Load Surcharge: δ =1.75 **LRFD [Table 3.4.1-1]**	-----------
Strength I (maximum)	-----------	Passive resistance of vertical elements: ϕ=0.75 **LRFD [Table11.5.7-1]**
Service I	-----------	Overall Stability: ϕ=0.75, when geotechnical parameters are well defined, and the slope does not support or contain a structural element
Service I	-----------	Overall Stability: ϕ=0.65, when geotechnical parameters are based on limited information, or the slope does support or contain a structural element

Table 14.10-1
Summary of Design Requirements

2. Foundation design parameters

   Use values provided by the Geotechnical Engineer of record for permanent sheet pile walls. Temporary sheet pile walls are the Contractor's responsibility.

3. Traffic surcharge

   - Traffic live load surcharge = 240 lb/ft^2 or determined by site condition.
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment

4. Retained soil

   - Unit weight = 120 lb/ft^3
   - Angle of internal friction as determined from the Geotechnical Report.

5. Soil pressure theory

Coulomb Theory.

6. Design life for anchorage hardware

   75 years minimum

7. Steel design properties

   Minimum yield strength = 39,000 psi

14.11 Soldier Pile Walls

Soldier pile walls are comprised of discrete vertical elements (usually steel H piles) and facing members (temporary and/or permanent) that extend between the vertical elements.

14.11.1 Design Procedure for Soldier Pile Walls

LRFD [11.8] Non-Gravity Cantilevered Walls covers the design of soldier pile walls. A simplified earth pressure distribution diagram is shown in **LRFD [3.11.5.6]** for permanent soldier pile walls. Another method that may be used is the "Conventional Method" or “Simplified Method” as described in "*United States Steel Sheet Piling Design Manual*", February, 1974. This method must be modified for the fact that it is based on continuous vertical wall elements whereas, soldier pile walls have discrete vertical wall elements. Using "Broms" method for designing drilled shafts is also acceptable.

The maximum spacing between vertical supporting elements (piles) depends on the wall height and the design parameters of the foundation soil. Spacing of 6 to 12 feet is typical. The piles are set in drilled holes and concrete is placed in the hole after the post is set. The pile system must be designed to handle maximum bending moment along length of embedded shaft. The maximum bending moment at any level in the facing can be determined from formulas in **LRFD [11.8.5.1]**. The minimum structural thickness on wall facings shall be 6 inches for precast panels and 10 inches with cast-in-place concrete.

The diameter of the drilled shaft is also dependent on the wall height and the design parameters of the foundation soil. The larger the diameter of the drilled shaft the smaller will be the required embedment of the shaft. The designer should try various shaft diameters to optimize the cost of the drilled shaft considering both material cost and drilling costs. Note that drilling costs are a function of both hole diameter and depth.

If the vertical elements are steel they shall be shop painted. Wall facings are usually given a special surface treatment created by brooming or tining vertically, using form liners, or using a pattern of rustication strips. The portion of the panel receiving the special treatment may be recessed, forming a border around the treated area. Concrete paints or stains may be used for color enhancements. When panel heights exceed 15 feet anchored walls may be needed. Anchored wall design must be in accordance with **LRFD [11.9]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

The concrete for soldier pile walls shall have a 28 day compressive strength of 4000 psi if non-prestressed and 5000 psi if prestressed except for the drilled shafts. Concrete for the drilled shafts shall have a 28 day compressive strength of 3500 psi. Reinforcement shall be uncoated Grade 60 in drilled shafts. In lieu of drainage aggregate a membrane may be used to seal the joints between the vertical elements and concrete panels to prevent water leakage. The front face of soldier pile walls shall be battered 1/4" per foot to account for short and long term deflection.

14.11.2 Summary of Design Requirements

Requirements

1. Resistance Factors

    - Overall Stability= 0.65 to 0.75 (based on how well defined the geotechnical parameters are and the support of structural elements)
    - Passive Resistance of vertical Elements = 0.75

2. Foundation Design Parameters

    Use values provided by the Geotechnical Engineer (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

3. Concrete Design Data

    - f'_c = 3500 psi (for drilled shafts)
    - f'_c = 4000 psi (non-prestressed panel)
    - f'_c = 5000 psi (prestressed panel)
    - f_y = 60,000 psi

4. Load Factors

    - Vertical earth pressure = 1.5
    - Lateral earth pressure = 1.5
    - Live load surcharge = 1.75

5. Traffic Surcharge

    - Traffic live load surcharge = 2 feet = 240 lb/ft^2
    - If no traffic surcharge, use 100 lb/ft^2

6. Retained Soil

    Use values provided by the Geotechnical Engineer (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

7. Soil Pressure Theory

    Rankine's Theory or Coulombs Theory at the discretion of the designer.

8. Design Life for Anchorage Hardware

   75 year minimum

9. Steel Design Properties (H-piles)

   Minimum yield strength = 50,000 psi

14.12 Temporary Shoring

This information is provided for guidance. Refer to the *Facilities Development Manual* for further details.

Temporary shoring is used to support a temporary excavation or protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Temporary shoring generally includes non-anchored temporary sheet piles, temporary soldier pile walls, temporary soil nails, cofferdam, or temporary mechanically stabilized earth (MSE) walls.

Temporary shoring is designed by the contractor. Shoring should not be required nor paid for when used primarily for the convenience of the contractor.

14.12.1 When Slopes Won't Work

Typically shoring will be required when safe slopes cannot be made due to geometric constraints of existing and proposed features within the available right-of-way. Occupation and Healthy Safety Administration (OSHA) requirements for temporary excavation slopes vary from a 1H:1V to a 2H:1V. The contractor is responsible for determining and constructing a safe slope based on actual site conditions.

In most cases, the designer can assume that an OSHA safe temporary slope can be cut on a 1.5H:1V slope; however other factors such as soil types, soil moisture, surface drainage, and duration of excavation should also be factored into the actual slope constructed. As an added safety factor, a 3-foot berm should be provided next to critical points or features prior to beginning a 1.5H:1V slope to the plan elevation of the proposed structure. Sufficient room should be provided adjacent to the structure for forming purposes (typically 2-3 feet).

14.12.2 Plan Requirements

Contract plans should schematically show in the plan and profile details all locations where the designer has determined that temporary shoring will be required. The plans should note the estimated length of the shoring as well as the minimum and maximum required height of exposed shoring. These dimensions will be used to calculate the horizontal projected surface area projected on a vertical plane of the exposed shoring face.

14.12.3 Shoring Design/Construction

The Contractor is responsible for design, construction, maintenance, and removal of the temporary shoring system in a safe and controlled manner. The adequacy of the design should be determined by a Wisconsin Professional Engineer knowledgeable of specific site conditions and requirements. The temporary shoring should be designed in accordance with the requirements described in 14.4.2 and 14.4.3. A signed and sealed copy of proposed designs must be submitted to the WisDOT Project Engineer for information.

14.13 Noise Barrier Walls

14.13.1 Wall Contract Process

WisDOT has classified all noise walls (both proprietary and non-proprietary) into three wall systems. All proprietary systems must be pre-approved prior to being considered for use on WisDOT projects. The three noise wall systems that are considered for WisDOT projects include the following:

1. Double-sided sound absorptive noise barriers
2. Single-sided sound absorptive noise barriers
3. Reflective noise barriers

If a wall is required, the designer must determine which wall system or systems are suitable for a given wall location. In some locations all wall systems may be suitable, whereas in other locations some wall systems may not be suitable. Information on aesthetic qualities and special finishes and colors of proprietary systems is available from the manufacturers. Information on approved concrete paints, stains and coatings is also available from the Structures Design Section. Designers are encouraged to contact the Structures Design Section (608-266-8494) if they have any questions about the material presented in the *Bridge Manual*.

The step by step process required to select a suitable wall system or systems for a given wall location is as follows:

Step 1: Investigate alternatives

Investigate alternatives to walls such as berms, plantings, etc.

Step 2: Geotechnical analysis

If a wall is required, geotechnical personnel shall conduct a soil investigation at the wall location and determine soil design parameters for the foundation soil. Geotechnical personnel are also responsible for recommending remedial methods of improving soil bearing capacity if required.

Step 3: Evaluate basic wall restrictions

The designer shall examine the list of suitable wall systems using the Geotechnical Report and remove any system that does not meet usage restrictions for the site.

Step 4: Determine suitable wall systems

The designer shall further examine the list of suitable wall systems for conformance to other considerations. Refer to Chapter 2 – General and Chapter 6 – Plan Preparation for a discussion on aesthetic considerations.

Step 5: Determine contract letting

After the designer has established the suitable wall system(s), the method of contract letting can be determined. The designer has several options based on the contents of the list.

Option 1:

The list contains only non-proprietary systems.

Under Option 1, the designer will furnish a complete design for one of the non-proprietary systems.

Option 2:

The list contains proprietary wall systems only or may contain both proprietary and non-proprietary wall systems, but the proprietary wall systems are deemed more appropriate than the non-proprietary systems.

Under Option 2 the designer will not furnish a design for any wall system. The contractor can build any wall system which is included on the list. The contractor is responsible for providing the complete design of the wall system selected, either by the wall supplier for proprietary walls or by the contractor's engineer for non-proprietary walls. Contract special provisions, if not in the Supplemental Specs., must be included in the contract document for each wall system that is allowed. Under Option 2, at least two and preferably three wall suppliers must have an approved product that can be used at the project site. See the *Facilities Development Manual* (Procedure 19-1-5) for any exceptions.

Option 3:

The list contains proprietary wall systems and non-proprietary wall systems and the non-proprietary systems are deemed equal or more appropriate than the proprietary systems.

Under Option 3 the designer will furnish a complete design for one of the non-proprietary systems, and list the other allowable wall systems.

Step 6: Prepare Contract Plans

Refer to section 14.16 for information required on the contract plans for proprietary systems. If a contractor chooses an alternate wall system, the contractor will provide the plans for the wall system chosen.

Step 7: Prepare Contract Special Provisions

The Structures Design Section and Region Offices have Special Provisions for each wall type and a generic Special Provision to be used for each project. The list of proprietary wall suppliers is maintained by the Materials Quality Assurance Unit.

Complete the generic Special Provision for the project by inserting the list of wall systems allowed and specifying the approved list of suppliers if proprietary wall systems are selected.

Step 8: Submit P.S.& E. (Plans, Specifications and Estimates)

When the plans are completed and all other data is completed, submit the project into the P.S.& E. process. Note that there is one bid item, square feet of exposed wall, for all wall quantities.

Step 9: Preconstruction Review

The contractor must supply the name of the wall system supplier and pertinent construction data to the project manager. This data must be accepted by the Office of Design, Contract Plans Section before construction may begin. Refer to the Construction and Materials Manual for specific details.

Step 10: Project Monitoring

It is the responsibility of the project manager to verify that the project is constructed with the previously accepted contract proposal. Refer to the Construction and Materials Manual for monitoring material certification, construction procedures and material requirements.

14.13.2 Pre-Approval Process

The purpose of the pre-approval process is to ascertain that a particular proprietary wall system has the capability of being designed and built according to the requirements and specifications of WisDOT. Any unique design requirements that may be required for a particular system are also identified during the pre-approval process. A design of a pre-approved system is acceptable for construction only after WisDOT has verified that the design is in accordance with the design procedures and criteria stated in the Certification Method of Acceptance for Noise Barrier Walls.

In addition to design criteria, suppliers must provide materials testing data and certification results for the required tests for durability, etc. The submittal requirements for the pre-approval process and other related information are available from the Materials Quality Assurance Unit, Madison, Wisconsin.

14.14 Contract Plan Requirements

The following minimum information shall be required on the plans.

1. Finish grades at rear and front of wall at 25 foot intervals or less.

2. Final cross sections as required for wall designer.

3. Beginning and end stations of wall and offsets from reference line to front face of walls. If reference line is a horizontal curve give offsets from a tangent to the curve.

4. Location of right-of-way boundaries, and construction easements relative to the front face of the walls.

5. Location of utilities if any and indicate whether to remain in place or be relocated or abandoned.

6. Special requirements on top of wall such as copings, railings, or traffic barriers.

7. Footing or leveling pad elevations if different than standard.

8. General notes on standard insert sheets.

9. Soil design parameters for retained soil, backfill soil and foundation soil including angle of internal friction, cohesion, coefficient of sliding friction, groundwater information and ultimate and/or allowable bearing capacity for foundation soil. If piles are required, give skin friction values and end bearing values for displacement piles and/or the allowable steel stress and anticipated driving elevation for end bearing piles.

10. Soil borings.

11. Details of special architectural treatment required for each wall system.

12. Wall systems, system or sub-systems allowed on projects.

13. Abutment details if wall is component of an abutment.

14. Connection and/or joint details where wall joins another structure.

15. Groundwater elevations.

16. Drainage provisions at heel of wall foundations.

17. Drainage at top of wall to divert run-off water.

18. Location of name plate.

14.15 Construction Documents

14.15.1 Bid Items and Method of Measurement

Proprietary retaining walls shall include all required bid items necessary to build the wall system provided by the contractor. The unit of measurement shall be square feet and shall include the exposed wall area between the footing and the top of the wall measured to the top of any copings. For setback walls the area shall be based on the walls projection on a vertical plane. The bid item includes designing the walls preparing plans, furnishing and placing all materials, including all excavations, temporary bracing, piling, (including delivered and driven), poured in place or precast concrete or blocks, leveling pads, soil reinforcement systems, structural steel, reinforcing steel, backfills and infills, drainage systems and aggregate, geotextiles, architectural treatment including painting and/or staining, drilled shafts, wall toppings unless excluded by contract, wall plantings, joint fillers, and all labor, tools, equipment and incidentals necessary to complete the work.

The contractor will be paid for the plan quantity as shown on the plans. (The intent is a lump sum bid item but is bid as square feet of wall). The top of wall coping is any type of cap placed on the wall. It does not include any barriers. Measurement is to the bottom of the barrier when computing exposed wall area.

Non-proprietary retaining walls are bid based on the quantity of materials used to construct the wall such as concrete, reinforcing steel, piling, etc. These walls are:

- Cast-in-Place Concrete Cantilever Walls
- Soldier Pile Walls
- Steel Sheet Piling Walls

14.15.2 Special Provisions

The Structures Design Section has Standard Special Provisions for:

- Wall Modular Block Gravity LRFD, Item SPV.0165.
- Wall Modular Block Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165
- Wall Concrete Panel Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165
- Wall CIP Facing Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Wire Faced Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165.
- Temporary Wall Wire Faced Mechcanically Stabilized Earth LRFD/QMP, Item SPV.0165
- *Wall Gabion LRFD, SPV under development.*

- *Wall Modular Bin or Crib LRFD, SPV under development.*

Note that the use of QMP Special Provisions began with the December 2014 letting or prior to December 2014 letting at the Region’s request.

The designer determines what wall systems(s) are applicable for the project. The approved names of suppliers are inserted for each eligible wall system. The list of approved proprietary wall suppliers is maintained by the Bureau of Structures which is responsible for the Approval Process for earth retaining walls, 14.16.

14.16 Submittal Requirements for Pre-Approval Process

14.16.1 General

The following four wall systems require the supplier or manufacturer to submit to the Structural Design Section a package that addresses the items specified in paragraph C.

1. Modular Block Gravity Walls

2. MSE Walls with Modular Block Facings

3. MSE Walls with Precast Concrete Panel Facings

4. Modular Concrete Bin or Crib Walls

14.16.2 General Requirements

Approval of retaining wall systems allows for use of these systems on Wisconsin Department of Transportation (WisDOT) projects upon the manufacturer's certification that the system as furnished to the contractor (or purchasing agency) complies with the design procedures specified in the *Bridge Manual*. WisDOT projects include: State, County and Municipal Federal Aid and authorized County and Municipal State Aid projects in addition to materials purchased directly by the state.

The manufacturer shall perform all specification tests with qualified personnel and maintain an acceptable quality control program. The manufacturer shall maintain records of all its control testing performed in the production of retaining wall systems. These test records shall be available at all times for examination by the Construction Materials Engineer for Highways or designee. Approval of materials will be contingent upon satisfactory compliance with procedures and material conformance to requirements as verified by source and field samples. Sampling will be performed by personnel during the manufacture of project specific materials.

14.16.3 Qualifying Data Required For Approval

Applicants requesting Approval for a specific system shall provide three copies of the documentation showing that they comply with *AASHTO LRFD* and *WisDOT Standard Specifications* and the design criteria specified in the *Bridge Manual*.

1. An overview of the system, including system theory.

2. Laboratory and field data supporting the theory.

3. Detailed design procedures, including sample calculations for installations with no surcharge, level surcharge and sloping surcharge.

4. Details of wall elements, analysis of structural elements, capacity demand ratio, load and resistance factors, estimated life, corrosion design procedure for soil reinforcement elements, procedures for field and laboratory evaluation including instrumentation and special requirements, if any.

5. Sample material and construction control specifications - showing material type, quality, certifications, field testing and placement procedures.

6. A well documented field construction manual describing in detail and with illustrations where necessary, the step by step construction sequence.

7. Details for mounting a concrete traffic barrier on the wall adjoining both concrete and flexible pavements (if applicable).

8. Pullout data for facing block/geogrid connection and soil pullout data (if applicable).

9. Submission of practical application with photos for all materials, surface textures and colors representative of products being certified.

10. Submission, if requested, to an on-site production process control review, and record keeping review.

11. List of installations including owner name and wall location.

12. Limitations of the wall system.

The above materials may be submitted at any time (recommend a minimum of 15 weeks) but, to be considered for a particular WisDOT project, must be approved prior to the bid opening date. The material should be clearly detailed and presented according to the prescribed outline.

After final review and approval of comments with the Bureau of Structures, the manufacturer will be approved to begin presenting the system on qualified projects.

14.16.4 Maintenance of Approval Status as a Manufacturer

The supplier or manufacturer must request to be reapproved bi-annually. The request shall be in writing and certify that the plant production process control and materials testing and design procedures haven't changed since the last review. The request shall be received within two years of the previous approval or the approval status will be terminated. Upon request for re-approval an on-site review of plant process control and materials testing may be conducted by WisDOT personnel. Travel expenses for trips outside the State of Wisconsin involved with this review will be borne by the manufacturer.

For periodic on-site reviews, access to the plant operations and materials records shall be provided to a representative of the Construction Materials Engineer during normal working hours upon request.

If the supplier or manufacturer introduces a new material, or cross-section, or a new design procedure, into its product line, the new feature must be submitted for approval. If the new feature/features are significantly different from the original product, the new product may be subjected to a complete review for approval.

14.16.5 Loss of Approved Status

Approval to deliver the approved system may be withdrawn under the following conditions:

Design Conformance

1. Construction does not follow design procedures.
2. Incorrect design procedures are used on projects.

Materials

3. Inability to consistently supply material meeting specification.
4. Inability to meet test method precision limits for quality control testing.
5. Lack of maintenance of required records.
6. Improper documentation of shipments.
7. Not maintaining an acceptable quality control program.

The decision to remove approval from a manufacturer on a specific system rests with the Construction Materials Engineer for Highways or the State Bridge Engineer.

14.17 References

1. State of Wisconsin, Department of Transportation, *Facilities Development Manual*

2. American Association of State highway and Transportation officials. *Standard Specification for highway Bridges*

3. American Association of State highway and Transportation officials. *AASHTO LRFD Bridge Design Specifications*

4. AASHTO LRFD Bridge Design Specification 4th Edition, 2007, AASHTO, 444 North Capitol Street, N.W., Suite 249, Washington, D.C. 20001.

5. Berg, Christopher and Samtani. *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. Publication No.FHWA-NHI-10-024.2009.

6. Bowles, Joseph E*. Foundation Analysis and Design 4th Edition.* McGraw Hill *1989*

7. Cudoto, Donald P. *Foundation Design Principles and Practices (* 2nd Edition*),* Prentice Halls

8. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls", 2302 Horse Pen Road, Herndon, Virginia 22071-3406.

9. Lazarte, Elias, Espinoza, Sabatini. *Geotechnical Engineering Circular No 7. Soil Nailing Walls*, FHWA

10. Publication No FHWA-SA-96-069R , "*Manual for design and construction of Soil Nail walls*

11. Publication No.FHWA-HI-98-032, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures".

12. Publication No.FHWA-NHI-07-071, "Earth retaining Structures".

13. Publication No.FHWA-NHI-09-083, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures".

14. Publication No. FHWA-NHI-09-087, "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced slopes"

15. Publication No.FHWA-NHI-10-024, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume I".

16. Publication No.FHWA-NHI-10-025, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume II".

14.18 Design Examples

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Table of Contents

E14-1 Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on a spread footing conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

Sample design calculations for bearing resistance, external stability (sliding, eccentricity and bearing) and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-1.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-1.1-1 will be designed appropriately to accommodate a State Trunk Highway. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

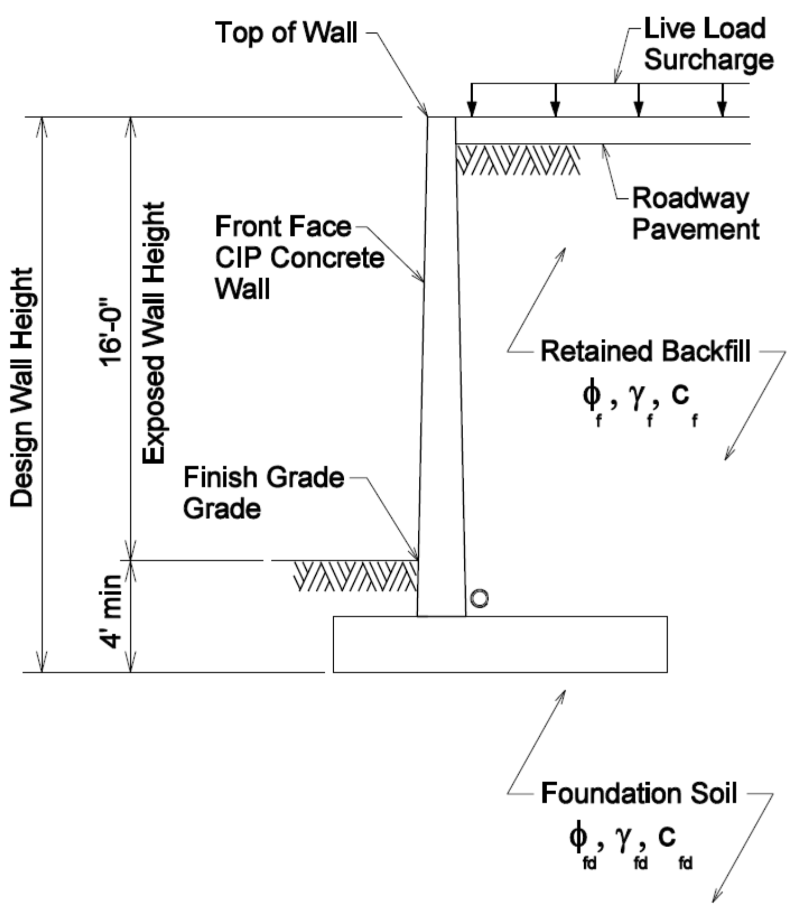

Figure E14-1.1-1
CIP Concrete Wall Adjacent to Highway

E14-1.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 30\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit weight, kcf

$c_f = 0$ Cohesion, ksf

$\delta = 21\ deg$ Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

Foundation Soil Design Parameters

$\phi_{fd} = 34\ deg$ Angle of internal friction

$\gamma_{fd} = 0.120$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, ksf

Reinforced Concrete Parameters

$f'_c = 3.5$ Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $w_c = \gamma_c$ Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ Modulus of elasticity of concrete, ksi **LRFD [C5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 1.0$ Distance from wall backface to edge of traffic, ft

$\frac{H}{2} = 10.00$ Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet)

Shall live load surcharge be included? check = "YES"

$h_{eq} = 2.0$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

Pavement Parameters

$\gamma_p = 0.150$ Pavement unit weight, kcf

Resistance Factors

$\phi_b = 0.55$ Bearing resistance (gravity and semi-gravity walls) **LRFD [Table 11.5.7-1]**

$\phi_s = 1.00$ Sliding resistance **LRFD [Table 11.5.7-1]**

$\phi_\tau = 1.00$ Sliding resistance (shear resistance between soil and foundation) **LRFD [Table 11.5.7-1]**

$\phi_{ep} = 0.50$ Sliding resistance (passive resistance) **LRFD [Table 10.5.5.2.2-1]**

$\phi_F = 0.90$ Concrete flexural resistance (Assuming tension-controlled) **LRFD [5.5.4.2.1]**

$\phi_v = 0.90$ Concrete shear resistance **LRFD [5.5.4.2.1]**

E14-1.3 Define Wall Geometry

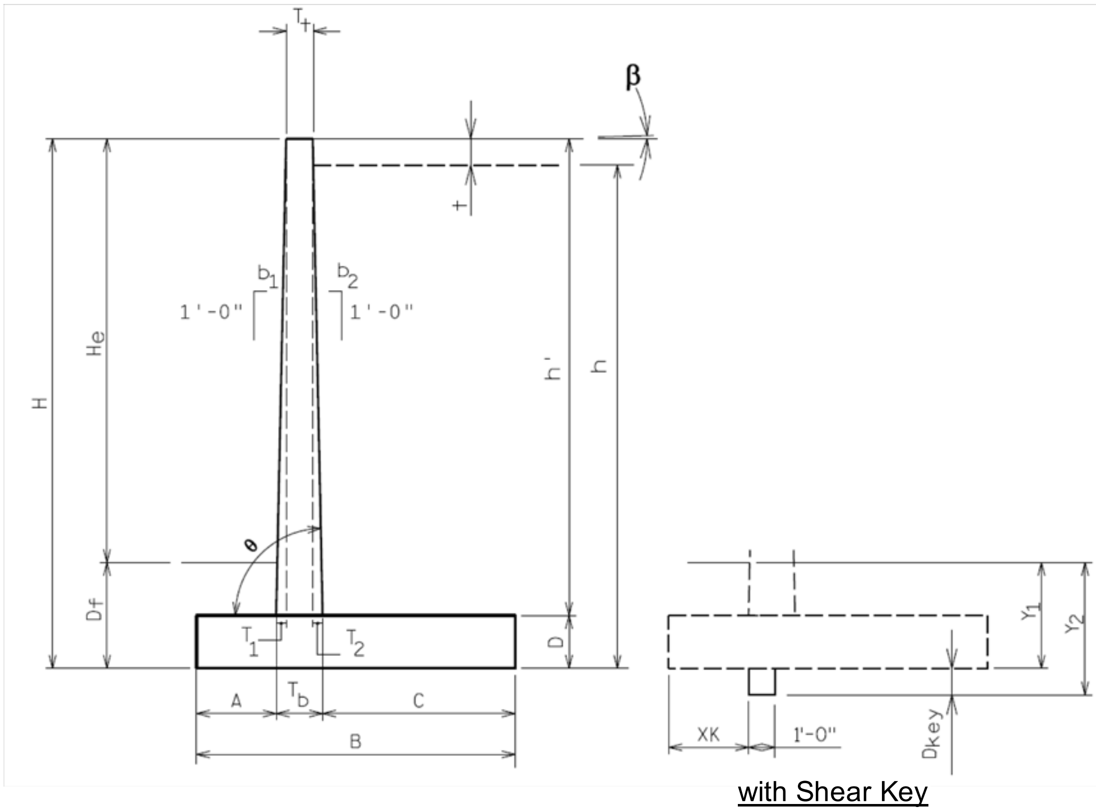

Figure E14-1.3-1
CIP Concrete Wall Geometry

Wall Geometry

$H_e = 16.0$	Exposed wall height, ft
$D_f = 4.0$	Footing cover, ft (WisDOT policy 4'-0" minimum)
$H = H_e + D_f$	Design wall height, ft
$T_t = 1.0$	Stem thickness at top of wall, ft
$b_1 = 0.25$	Front wall batter, in/ft (b_1H:12V)
$b_2 = 0.50$	Back wall batter, in/ft (b_2H:12V)
$\beta = 0$ deg	Inclination of ground slope behind face of wall, deg (horizontal)
$t = 1.0$	Pavement thickness, ft

Preliminary Wall Dimensioning

Selecting the most optimal wall configuration is an iterative process and depends on site conditions, cost considerations, wall geometry and aesthetics. For this example, the iterative process has been completed and the final wall dimensions are used for design checks.

$H = 20.0$	Design wall height, ft	
$B = 10.0$	Footing base width, ft (2/5H to 3/5H)	
$A = 3.5$	Toe projection, ft (H/8 to H/5)	
$D = 2.0$	Footing thickness, ft (H/8 to H/5)	
	WisDOT policy: $H \leq 10'\text{-}0"$ $D_{min} = 1'\text{-}6"$ $H > 10'\text{-}0"$ $D_{min} = 2'\text{-}0"$	

Shear Key Dimensioning

$D_{key} = 1.0$	Depth of shear key from bottom of footing, ft	
$D_w = 1.0$	Width of shear key, ft	
$XK = A$	Distance from toe to shear key, ft	

Other Wall Dimensioning

$h' = H - D$	Stem height, ft	$\boxed{h' = 18.00}$
$T_1 = b_1 \dfrac{h'}{12}$	Stem front batter width, ft	$\boxed{T_1 = 0.375}$
$T_2 = b_2 \dfrac{h'}{12}$	Stem back batter width, ft	$\boxed{T_2 = 0.750}$
$T_b = T_1 + T_t + T_2$	Stem thickness at bottom of wall, ft	$\boxed{T_b = 2.13}$
$C = B - A - T_b$	Heel projection, ft	$\boxed{C = 4.38}$
$\theta = \text{atan}\left(\dfrac{12}{b_2}\right)$	Angle of back face of wall to horizontal	$\boxed{\theta = 87.6\ \text{deg}}$
$b = 12$	Concrete strip width for design, in	
$y_1 = D_f$	Bottom of footing depth, ft	$\boxed{y_1 = 4.0}$
$y_2 = D_f + D_{key}$	Bottom of shear key depth, ft	$\boxed{y_2 = 5.0}$
$h = H - t + (T_2 + C)\tan(\beta)$	Retained soil height, ft	$\boxed{h = 19.0}$

E14-1.4 Permanent and Transient Loads

In this example, load types DC (dead load components), EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used. Soil above the toe will be ignored as well as its passive resistance. When a shear key is present only the passive soil resistance from the vertical face of the shear key will be included in sliding resistance.

E14-1.4.1 Compute Earth Pressure Coefficients

Active and passive earth pressures

E14-1.4.1.1 Compute Active Earth Pressure Coefficient

Compute the coefficient of active earth pressure using Coulomb Theory **LRFD [Eq 3.11.5.3-1]**

$\phi_f = 30.0 \text{ deg}$

$\beta = 0.0 \text{ deg}$

$\theta = 87.6 \text{ deg}$

$\delta = 21.0 \text{ deg}$

$k_a =$

$$\frac{\sin(\theta + \phi_f)^2}{\Gamma \sin(\theta)^2 \sin(\theta - \delta)}$$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2 \qquad \boxed{\Gamma = 2.726}$$

$$k_a = \frac{\sin(\theta + \phi_f)^2}{\Gamma \sin(\theta)^2 \sin(\theta - \delta)} \qquad \boxed{k_a = 0.314}$$

E14-1.4.1.2 Compute Passive Earth Pressure Coefficient

Compute the coefficient of passive earth pressure using Rankine Theory

$$k_p = \tan\left(45 \text{ deg} + \frac{\phi_{fd}}{2}\right)^2 \qquad \boxed{k_p = 3.54}$$

E14-1.4.2 Compute Unfactored Loads

The forces and moments are computed by using Figures E14-1.3-1 and E14-1.3-3 and by their respective load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**

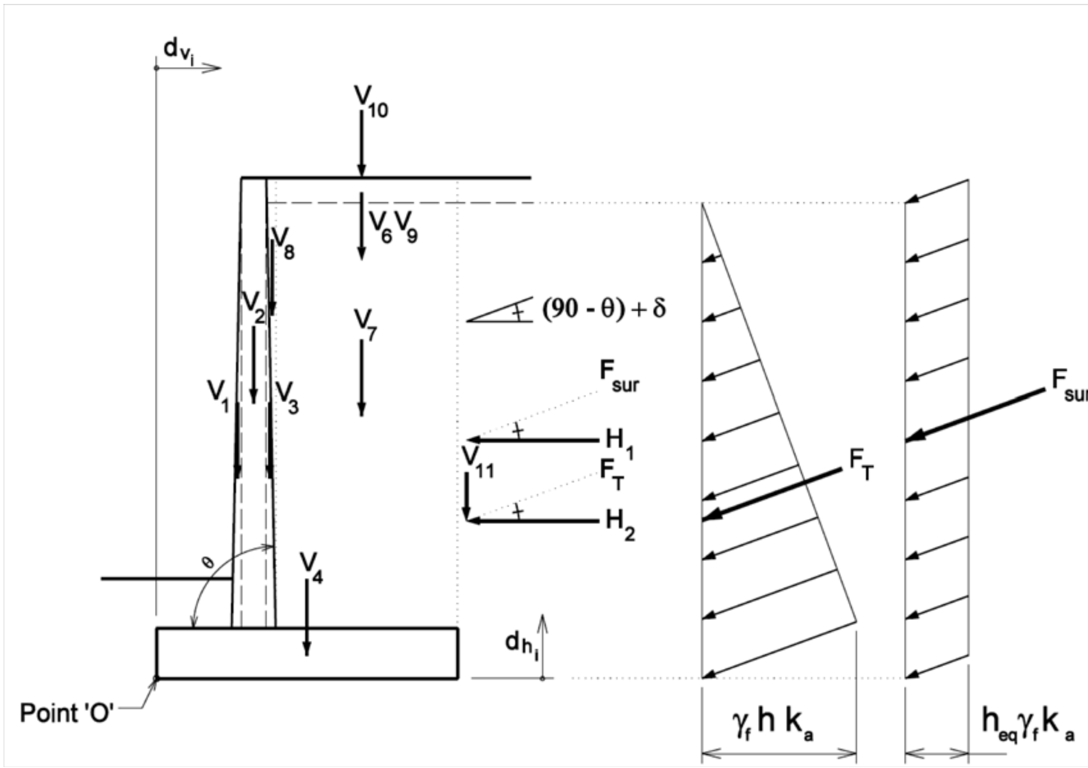

Figure E14-1.4-3
CIP Concrete Wall - External Stability

Active Earth Force Resultant (kip/ft), F_T

$$F_T = \frac{1}{2}\gamma_f h^2 k_a$$ Active earth force resultant (EH) $\boxed{F_T = 6.81}$

Live Load Surcharge Load (kip/ft), F_{sur}

$$F_{sur} = \gamma_f h_{eq} h k_a$$ Live load surcharge resultant (LS) $\boxed{F_{sur} = 1.43}$

Vertical Loads (kip/ft), V_i

$$V_1 = \frac{1}{2} T_1 h' \gamma_c$$ Wall stem front batter (DC) $\boxed{V_1 = 0.51}$

$$V_2 = T_t h' \gamma_c$$ Wall stem (DC) $\boxed{V_2 = 2.70}$

$$V_3 = \frac{1}{2} T_2 h' \gamma_c$$ Wall stem back batter (DC) $\boxed{V_3 = 1.01}$

$V_4 = D\ B\ \gamma_c$ Wall footing (DC) $\boxed{V_4 = 3.00}$

$V_6 = t\left(T_2 + C\right)\gamma_p$ Pavement (DC) $\boxed{V_6 = 0.77}$

$V_7 = C\ (h' - t)\ \gamma_f$ Soil backfill - heel (EV) $\boxed{V_7 = 8.92}$

$V_8 = \frac{1}{2}\ T_2\ (h' - t)\ \gamma_f$ Soil backfill - batter (EV) $\boxed{V_8 = 0.77}$

$V_9 = \frac{1}{2}\left(T_2 + C\right)\left[\left(T_2 + C\right)\tan(\beta)\right]\gamma_f$

Soil backfill - backslope (EV) $\boxed{V_9 = 0.00}$

$V_{10} = h_{eq}\left(T_2 + C\right)\gamma_f$

Live load surcharge (LS) $\boxed{V_{10} = 1.23}$

$V_{11} = F_T\ \sin(90\ \text{deg} - \theta + \delta)$

Active earth force resultant (vertical component - EH) $\boxed{V_{11} = 2.70}$

Moments produced from vertical loads about Point 'O' (kip-ft/ft), MV_i

Moment Arm (ft)		Moment (kip-ft/ft)	
$d_{v1} = A + \frac{2}{3}T_1$	$\boxed{d_{v1} = 3.8}$	$MV_1 = V_1\ d_{v1}$	$\boxed{MV_1 = 1.9}$
$d_{v2} = A + T_1 + \frac{T_t}{2}$	$\boxed{d_{v2} = 4.4}$	$MV_2 = V_2\ d_{v2}$	$\boxed{MV_2 = 11.8}$
$d_{v3} = A + T_1 + T_t + \frac{T_2}{3}$	$\boxed{d_{v3} = 5.1}$	$MV_3 = V_3\ d_{v3}$	$\boxed{MV_3 = 5.2}$
$d_{v4} = \frac{B}{2}$	$\boxed{d_{v4} = 5.0}$	$MV_4 = V_4\ d_{v4}$	$\boxed{MV_4 = 15.0}$
$d_{v6} = B - \left(\frac{T_2 + C}{2}\right)$	$\boxed{d_{v6} = 7.4}$	$MV_6 = V_6\ d_{v6}$	$\boxed{MV_6 = 5.7}$
$d_{v7} = B - \frac{C}{2}$	$\boxed{d_{v7} = 7.8}$	$MV_7 = V_7\ d_{v7}$	$\boxed{MV_7 = 69.7}$

$$d_{v8} = A + T_1 + T_t + \frac{2T_2}{3} \qquad \boxed{d_{v8} = 5.4} \qquad MV_8 = V_8\, d_{v8} \qquad \boxed{MV_8 = 4.1}$$

$$d_{v9} = A + T_1 + T_t + \frac{2\left(T_2 + C\right)}{3} \qquad \boxed{d_{v9} = 8.3} \qquad MV_9 = V_9\, d_{v9} \qquad \boxed{MV_9 = 0.0}$$

$$d_{v10} = B - \left(\frac{T_2 + C}{2}\right) \qquad \boxed{d_{v10} = 7.4} \qquad MV_{10} = V_{10}\, d_{v10} \qquad \boxed{MV_{10} = 9.1}$$

$$d_{v11} = B \qquad \boxed{d_{v11} = 10.0} \qquad MV_{11} = V_{11}\, d_{v11} \qquad \boxed{MV_{11} = 27.0}$$

Horizontal Loads (kip/ft), H_i

$$H_1 = F_{sur} \cos(90\text{ deg} - \theta + \delta)$$

Live load surcharge (LS) $\boxed{H_1 = 1.32}$

$$H_2 = F_T \cos(90\text{ deg} - \theta + \delta)$$

Active earth force
(horizontal component) (EH) $\boxed{H_2 = 6.25}$

Moments produced from horizontal loads about about Point 'O' (kip-ft/ft), MH_i

Moment Arm (ft) | Moment (kip-ft/ft)

$$d_{h1} = \frac{h}{2} \qquad \boxed{d_{h1} = 9.5} \qquad MH_1 = H_1\, d_{h1} \qquad \boxed{MH_1 = 12.5}$$

$$d_{h2} = \frac{h}{3} \qquad \boxed{d_{h2} = 6.3} \qquad MH_2 = H_2\, d_{h2} \qquad \boxed{MH_2 = 39.6}$$

Summary of Unfactored Forces & Moments:

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Wall stem front batter	0.51	d_{v1}	3.8	MV_1	1.9	DC
V_2	Wall stem	2.70	d_{v2}	4.4	MV_2	11.8	DC
V_3	Wall stem back batter	1.01	d_{v3}	5.1	MV_3	5.2	DC
V_4	Wall footing	3.00	d_{v4}	5.0	MV_4	15.0	DC
V_6	Pavement	0.77	d_{v6}	7.4	MV_6	5.7	DC
V_7	Soil backfill	8.92	d_{v7}	7.8	MV_7	69.7	EV
V_8	Soil backfill	0.77	d_{v8}	5.4	MV_8	4.1	EV
V_9	Soil backfill	0.00	d_{v9}	8.3	MV_9	0.0	EV
V_{10}	Live load surcharge	1.23	d_{v10}	7.4	MV_{10}	9.2	LS
V_{11}	Active earth pressure	2.70	d_{v11}	10.0	MV_{11}	27.0	EH

Table E14-1.4-1
Unfactored Vertical Forces & Moments

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Live load surcharge	1.32	d_{h1}	9.5	MH_1	12.5	LS
H_2	Active earth force	6.25	d_{h2}	6.3	MH_2	39.6	EH

Table E14-1.4-2
Unfactored Horizontal Forces & Moments

E14-1.4.3 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all the load modifiers to zero($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be used in this example:

Load Combination	γ_{DC}	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	0.90	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	Bearing, Wall Strength
Service I	1.00	1.00	1.00	1.00	1.00	Wall Crack Control

Table E14-1.4-3
Load Combinations

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)}$ = 0.9, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Vertical loads from vehicle collision need not be applied with transverse loads. By inspection, transverse loads will control Extreme Event Load Combination for this example.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_{10}\gamma_{EH(max)}$ and $H_2\gamma_{EH(max)}$ or $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(min)}$, not $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(max)}$.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-1.4.4 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$$V_{DC} = V_1 + V_2 + V_3 + V_4 + V_6 \qquad \boxed{V_{DC} = 8.0}$$

$$V_{EV} = V_7 + V_8 + V_9 \qquad \boxed{V_{EV} = 9.7}$$

$$V_{LS} = V_{10} \qquad \boxed{V_{LS} = 1.2}$$

$$V_{EH} = V_{11} \qquad \boxed{V_{EH} = 2.7}$$

$$H_{LS} = H_1 \qquad \boxed{H_{LS} = 1.3}$$

$$H_{EH} = H_2 \qquad \boxed{H_{EH} = 6.3}$$

Unfactored moments by load type (kip-ft/ft)

$$M_{DC} = MV_1 + MV_2 + MV_3 + MV_4 + MV_6 \qquad \boxed{M_{DC} = 39.6}$$

$$M_{EV} = MV_7 + MV_8 + MV_9 \qquad \boxed{M_{EV} = 73.8}$$

$$M_{LS1} = MV_{10} \qquad \boxed{M_{LS1} = 9.1}$$

$$M_{EH1} = MV_{11} \qquad \boxed{M_{EH1} = 27.0}$$

$$M_{LS2} = MH_1 \qquad \boxed{M_{LS2} = 12.5}$$

$$M_{EH2} = MH_2 \qquad \boxed{M_{EH2} = 39.6}$$

Factored vertical loads by limit state (kip/ft)

$$V\_Ia = n\left(0.90V_{DC} + 1.00V_{EV} + 0.00\ V_{LS} + 1.50\ V_{EH}\right) \qquad \boxed{V\_Ia = 20.9}$$

$$V\_Ib = n\left(1.25V_{DC} + 1.35V_{EV} + 1.75\ V_{LS} + 1.50\ V_{EH}\right) \qquad \boxed{V\_Ib = 29.3}$$

$$V\_Ser = n\left(1.00V_{DC} + 1.00V_{EV} + 1.00\ V_{LS} + 1.00\ V_{EH}\right) \qquad \boxed{V\_Ser = 21.6}$$

Factored horizontal loads by limit state (kip/ft)

$H\_Ia = n(1.75H_{LS} + 1.50H_{EH})$ $\boxed{H\_Ia = 11.7}$

$H\_Ib = n(1.75H_{LS} + 1.50H_{EH})$ $\boxed{H\_Ib = 11.7}$

$H\_Ser = n(1.00H_{LS} + 1.00H_{EH})$ $\boxed{H\_Ser = 7.6}$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$MV\_Ia = n(0.90M_{DC} + 1.00M_{EV} + 0.00M_{LS1} + 1.50\ M_{EH1})$ $\boxed{MV\_Ia = 150.0}$

$MV\_Ib = n(1.25M_{DC} + 1.35M_{EV} + 1.75M_{LS1} + 1.50\ M_{EH1})$ $\boxed{MV\_Ib = 205.8}$

$MV\_Ser = n(1.00M_{DC} + 1.00M_{EV} + 1.00M_{LS1} + 1.00\ M_{EH1})$ $\boxed{MV\_Ser = 149.6}$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$MH\_Ia = n(1.75M_{LS2} + 1.50\ M_{EH2})$ $\boxed{MH\_Ia = 81.3}$

$MH\_Ib = n(1.75M_{LS2} + 1.50\ M_{EH2})$ $\boxed{MH\_Ib = 81.3}$

$MH\_Ser = n(1.00M_{LS2} + 1.00\ M_{EH2})$ $\boxed{MH\_Ser = 52.1}$

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	20.9	150.0	11.7	81.3
Strength Ib	29.3	205.8	11.7	81.3
Service I	21.6	149.6	7.6	52.1

Table E14-1.4-4
Summary of Factored Loads & Moments

E14-1.5 Compute Bearing Resistance, q_R

Nominal bearing resistance, q_n **LRFD [Eq 10.6.3.1.2a-1]**

$$q_n = c_{fd} N_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B' N_{\gamma m} C_{w\gamma}$$

Compute the resultant location (distance from Point 'O' Figure E14-4.4-3)

$\Sigma M_R = MV\_Ib$ $\boxed{\Sigma M_R = 205.8}$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ $\boxed{\Sigma M_O = 81.3}$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ $\boxed{\Sigma V = 29.3}$ Summation of vertical loads for Strength Ib

$x = \dfrac{\Sigma M_R - \Sigma M_O}{\Sigma V}$ Distance from Point "O" the resultant intersects the base $\boxed{x = 4.25}$ ft

Compute the wall eccentricity

$e = \dfrac{B}{2} - x$ $\boxed{e = 0.75}$ ft

Define the foundation layout

$B' = B - 2\,e$ Footing width $\boxed{B' = 8.5}$ ft

$L' = 90.0$ Footing length (Assumed) $\boxed{L' = 90.0}$ ft

$H' = H\_Ib$ Summation of horizontal loads for Strength Ib $\boxed{H' = 11.7}$ kip/ft

$V' = V\_Ib$ Summation of vertical loads for Strength Ib $\boxed{V' = 29.3}$ kip/ft

$D_f = 4.00$ Footing embedment

$\theta' = 90deg$ Direction of H' and V' resultant measured from wall backface **LRFD [Figure C10.6.3.1.2a-1]** $\boxed{\theta' = 90.0\ deg}$

Compute bearing capacity factors per **LRFD [Table 10.6.3.1.2a-1]**

$\phi_{fd} = 34.0\ deg$ $N_q = 29.4$ $N_c = 42.2$ $N_\gamma = 41.1$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**

Since the friction angle, ϕ_f, is > 0 the following equations are used:

$s_c = 1 + \left(\dfrac{B'}{L'}\right)\left(\dfrac{N_q}{N_c}\right)$ $\boxed{s_c = 1.07}$

$s_q = 1 + \left(\dfrac{B'}{L'}\tan(\phi_{fd})\right)$ $\boxed{s_q = 1.06}$

$s_\gamma = 1 - 0.4\left(\dfrac{B'}{L'}\right)$ $\boxed{s_\gamma = 0.96}$

Compute load inclination factors using **LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]**

$$n = \frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}} \cos(\theta')^2 + \frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}} \sin(\theta')^2 \qquad \boxed{n = 1.91}$$

$$i_q = \left(1 - \frac{H'}{V' + c_{fd} B' L' \frac{1}{\tan(\phi_{fd})}}\right)^n \qquad \boxed{i_q = 0.38}$$

$$i_\gamma = \left(1 - \frac{H'}{V' + c_{fd} B' L' \frac{1}{\tan(\phi_{fd})}}\right)^{n+1} \qquad \boxed{i_\gamma = 0.23}$$

$$i_c = i_q - \left(\frac{1 - i_q}{N_q - 1}\right) \qquad \text{For } \phi_{fd} > 0: \qquad \boxed{i_c = 0.36}$$

Note: The use of load inclination factors shall be determined by the engineer.

Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. While it can be assumed that the soils above the footing are as competent as beneath the footing, the depth correction factor is taken as 1.0 since D_f/B is less than 1.0.

$$d_q = 1.00$$

Determine coefficients C_{wq} and $C_{w\gamma}$ assuming that the water depth is greater than 1.5 times the footing base plus the embedment depth per **LRFD [Table 10.6.3.1.2a-2]**

$$C_{wq} = 1.0 \qquad \text{where } D_w > 1.5B + D_f$$

$$C_{w\gamma} = 1.0 \qquad \text{where } D_w > 1.5B + D_f$$

Compute modified bearing capacity factors
LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]

$$N_{cm} = N_c\, s_c\, i_c \qquad \boxed{N_{cm} = 16.0}$$

$$N_{qm} = N_q\, s_q\, d_q\, i_q \qquad \boxed{N_{qm} = 11.8}$$

$$N_{\gamma m} = N_\gamma\, s_\gamma\, i_\gamma \qquad \boxed{N_{\gamma m} = 9.0}$$

Compute nominal bearing resistance, q_n, **LRFD [Eq 10.6.3.1.2a-1]**

$$q_n = c_{fd} N_{cm} + \gamma_{fd} D_f N_{qm} C_{wq} + 0.5\, \gamma_{fd} B' N_{\gamma m} C_{w\gamma} \qquad \boxed{q_n = 10.25} \text{ ksf/ft}$$

Compute factored bearing resistance, q_R, **LRFD [Eq 10.6.3.1.1]**

$$\phi_b = 0.55$$

$$q_R = \phi_b\, q_n \qquad \boxed{q_R = 5.64} \text{ ksf/ft}$$

E14-1.6 Evaluate External Stability of Wall

Three potential external failure mechanisms will be considered in this example. These failures include bearing, limiting eccentricity and sliding. Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-1.6.1 Bearing Resistance at Base of the Wall

The following calculations are based on **Strength Ib**:

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

$\Sigma M_R = MV\_Ib$ $\quad\Sigma M_R = 205.8$ kip-ft/ft

$\Sigma M_O = MH\_Ib$ $\quad\Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = V\_Ib$ $\quad\Sigma V = 29.3$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ Distance from Point "O" the resultant intersects the base

$\boxed{x = 4.25}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x$$ $\boxed{e = 0.75}$ ft

Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. When the foundation eccentricity is negative the actual bearing width, B, will be used.

Compute the ultimate bearing stress

$$\sigma_V = \frac{\Sigma V}{B - 2\,e}$$ $\boxed{\sigma_V = 3.44}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 5.64}$ ksf/ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Bearing1} = \frac{q_R}{\sigma_V}$$ $\boxed{CDR_{Bearing1} = 1.64}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-1.6.2 Limiting Eccentricity at Base of the Wall

The location of the resultant of the reaction forces is limited to the middle two-thirds o base width for a soil foundation (i.e., e_{max} = B/3). The following calculations are base

Strength Ia:

Maximum eccentricity

$$e_{max} = \frac{B}{3}$$ $\boxed{e_{max} = 3.33}$ ft

Compute resultant location (distance from Point 'O' Figure E14-1.4.3)

$\Sigma M_R = MV\_Ia$ $\Sigma M_R = 150.0$ kip-ft/ft

$\Sigma M_O = MH\_Ia$ $\Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = V\_Ia$ $\Sigma V = 20.9$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ Distance from Point "O" the resultant intersects the base

$\boxed{x = 3.29}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x$$ $\boxed{e = 1.71}$ ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Eccentricity1} = \frac{e_{max}}{e}$$ $\boxed{CDR_{Eccentricity1} = 1.94}$

Is the $CDR \geq 1.0$? $\boxed{check = "OK"}$

E14-1.6.3 Sliding Resistance at Base of the Wall

For sliding failure, the horizontal force effects, R_u, is checked against the sliding resistance, R_R, where $R_R = \phi R_n$ **LRFD [10.6.3.4]**. If sliding resistance is not adequate a shear key will be investigated. The following calculations are based on **Strength Ia**:

Factored Sliding Force, R_u

$R_u = H\_Ia$ $\boxed{R_u = 11.7}$ kip/ft

Sliding Resistance, R_R

$$R_R = \phi_s R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

Compute sliding resistance between soil and foundation, $\phi_T R_T$

$\Sigma V = V\_Ia$ $\boxed{\Sigma V = 20.9}$ kip/ft

$R_T = \Sigma V \tan(\phi_{fd})$ $\boxed{R_T = 14.1}$ kip/ft

$\phi_T = 1.00$ $\boxed{\phi_T R_T = 14.1}$ kip/ft

Compute passive resistance throughout the design life of the wall, $\phi_{ep} R_{ep}$

$r_{ep1} = k_p \gamma_{fd} y_1$ Nominal passive pressure at y_1 $\boxed{r_{ep1} = 1.70}$ kip/ft

$r_{ep2} = k_p \gamma_{fd} y_2$ Nominal passive pressure at y_2 $\boxed{r_{ep2} = 2.12}$ kip/ft

$R_{ep} = \dfrac{r_{ep1} + r_{ep2}}{2}(y_2 - y_1)$ $\boxed{R_{ep} = 1.9}$ kip/ft

$\phi_{ep} = 0.50$ $\boxed{\phi_{ep} R_{ep} = 1.0}$ kip/ft

Compute nominal resistance against failure by sliding, R_n

$R_n = \phi_T R_T + \phi_{ep} R_{ep}$ $\boxed{R_n = 15.1}$ kip/ft

Compute factored resistance against failure by sliding, R_R

$\phi_s = 1.00$

$R_R = \phi_s R_n$ $\boxed{R_R = 15.1}$ kip/ft

<u>Capacity:Demand Ratio (CDR)</u>

$CDR_{Sliding1} = \dfrac{R_R}{R_u}$ $\boxed{CDR_{Sliding1} = 1.29}$

Is the CDR ≥ 1.0 ? $\boxed{check = "OK"}$

E14-1.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. The critical sections for flexure are taken at the front, back and bottom of them stem. For simplicity, critical sections for shear will be taken at the critical sections used for flexsure. In actuality, the toe and stem may be designed for shear at the effective depth away from the face. Crack control and temperature and shrinkage considerations will also be included.

E14-1.7.1 Evaluate Heel Strength

Analyze heel requirements.

E14-1.7.1.1 Evaluate Heel Shear Strength

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B}V_4 + V_6\right) + 1.35\left(V_7 + V_8 + V_9\right) + 1.75\left(V_{10}\right) + 1.50\left(V_{11}\right)$$

$\boxed{V_u = 21.9}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\,\beta\lambda\sqrt{f'_c}\,b_v\,d_v$

$V_{n2} = 0.25\,f'_c\,b_v\,d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 7.0$ in (bar spacing)

$Bar_{No} = 6$ (transverse bar size)

$Bar_D = 0.750$ in (transverse bar diameter)

$Bar_A = 0.440$ in² (transverse bar area)

$\alpha_1 = 0.85$ (for $f'_c \leq 10.0$ ksi) **LRFD [5.7.2.2]**

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$

$\boxed{A_s = 0.75}$ in²/ft

$$d_s = D\,12 - cover - \frac{Bar_D}{2}$$

$\boxed{d_s = 21.6}$ in

$$a = \frac{A_s\,f_y}{\alpha_1\,f'_c\,b}$$

$\boxed{a = 1.3}$ in

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 21.0} \text{ in}$$

$$d_{v2} = 0.9\, d_s \qquad \boxed{d_{v2} = 19.5} \text{ in}$$

$$d_{v3} = 0.72\, D\, 12 \qquad \boxed{d_{v3} = 17.3} \text{ in}$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 21.0} \text{ in}$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_c = 0.0316\, \beta\, \lambda \sqrt{f'_c}\, b\, d_v \qquad \boxed{V_c = 29.8} \text{ kip/ft}$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 29.8} \text{ kip/ft}$$

$$V_{n2} = 0.25\, f'_c\, b\, d_v \qquad \boxed{V_{n2} = 220.4} \text{ kip/ft}$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 29.8} \text{ kip/ft}$$

$$V_r = \phi_v\, V_n \qquad \boxed{V_r = 26.8} \text{ kip/ft}$$

$$\boxed{V_u = 21.9} \text{ kip/ft}$$

Is V_u less than V_r? $\boxed{\text{check} = \text{"OK"}}$

E14-1.7.1.2 Evaluate Heel Flexural Strength

$V_u = 21.9$ kip/ft

$$M_u = V_u \frac{C}{2} \qquad \boxed{M_u = 47.9} \text{ kip-ft/ft}$$

Calculated the capacity of the heel in flexure at the face of the stem:

$$M_n = A_s\, f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12} \qquad \boxed{M_n = 79.2} \text{ kip-ft/ft}$$

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$$c = \frac{a}{\beta_1} \qquad \boxed{c = 1.49} \text{ in}$$

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Note: if $\phi_F = 0.75$ Section is compression-controlled

if $0.75 < \phi_F < 0.90$ Section is in transition

if $\phi_F = 0.90$ Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F\, M_n$$

$\boxed{M_r = 71.2}$ kip-ft/ft

$\boxed{M_u = 47.9}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\,\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.449}$ ksi

$$I_g = \frac{1}{12}\, b\,(D\ 12)^3$$ $\boxed{I_g = 13824}$ in^4

$$y_t = \frac{1}{2}\, D\ 12$$ $\boxed{y_t = 12.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1152}$ in^3

$M_{cr} = \gamma_3\left(\gamma_1\, f_r\right) S_c$ therefore, $M_{cr} = 1.1\, f_r\, S_c$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$$ $\boxed{M_{cr} = 47.4}$ kip-ft/ft

$1.33\ M_u = 63.7$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33{*}M_u$? check = "OK"

E14-1.7.2 Evaluate Toe Strength

The structural design of the footing toe is calculated using a linear contact stress distribution for bearing for all soil and rock conditions.

E14-1.7.2.1 Evaluate Toe Shear Strength

For **Strength Ib**:

$\Sigma M_R = MV\_Ib$ $\Sigma M_R = 205.8$ kip-ft/ft

$\Sigma M_O = MH\_Ib$ $\Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = V\_Ib$ $\Sigma V = 29.3$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ $x = 4.3$ ft

$$e = \max\left(0, \frac{B}{2} - x\right)$$ $e = 0.75$ ft

$$\sigma_{max} = \frac{\Sigma V}{B}\left(1 + 6\frac{e}{B}\right)$$ $\sigma_{max} = 4.24$ ksf/ft

$$\sigma_{min} = \frac{\Sigma V}{B}\left(1 - 6\frac{e}{B}\right)$$ $\sigma_{min} = 1.62$ ksf/ft

Calculate the average stress on the toe

$$\sigma_v = \frac{\sigma_{max} + \left[\sigma_{min} + \frac{B - A}{B}\left(\sigma_{max} - \sigma_{min}\right)\right]}{2}$$ $\sigma_v = 3.78$ ksf/ft

$V_u = \sigma_v A$ $V_u = 13.2$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

in which: $V_c = 0.0316\ \beta\lambda\sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Design footing toe for shear

$cover = 3.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 5$ (transverse bar size)

$Bar_D = 0.63$ in (transverse bar diameter)

$Bar_A = 0.31$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 0.41} \ in^2/ft$$

$$d_s = D\ 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 20.7} \ in$$

$$a = \frac{A_s\ f_y}{\alpha_1\ f'_c\ b} \qquad \boxed{a = 0.7} \ in$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 20.3} \ in$$

$$d_{v2} = 0.9\ d_s \qquad \boxed{d_{v2} = 18.6} \ in$$

$$d_{v3} = 0.72\ D\ 12 \qquad \boxed{d_{v3} = 17.3} \ in$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 20.3} \ in$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_c = 0.0316\ \beta\ \lambda \sqrt{f'_c}\ b\ d_v \qquad \boxed{V_c = 28.9} \ kip/ft$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 28.9} \ kip/ft$$

$$V_{n2} = 0.25\ f'_c\ b\ d_v \qquad \boxed{V_{n2} = 213.6} \ kip/ft$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 28.9} \ kip/ft$$

$$V_r = \phi_v\ V_n \qquad \boxed{V_r = 26.0} \ kip/ft$$

$$\boxed{V_u = 13.2} \ kip/ft$$

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-1.7.2.2 Evaluate Toe Flexural Strength

$V_u = 13.2$ kip/ft

$$M_u = V_u \frac{A}{2}$$ $\boxed{M_u = 23.2}$ kip-ft/ft

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 42.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 0.82}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F M_n$$ $\boxed{M_r = 37.8}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.449}$ ksi

$$I_g = \frac{1}{12} b\,(D\,12)^3$$ $\boxed{I_g = 13824}$ in^4

$$y_t = \frac{1}{2} D\,12$$ $\boxed{y_t = 12.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1152}$ in^3

$M_{cr} = 1.1\ f_r\ S_c\ \frac{1}{12}$ from E14-1.7.1.2 $\boxed{M_{cr} = 47.4}$ kip-ft/ft

$\boxed{1.33\ M_u = 30.8}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33{*}M_u$?

$\boxed{check = \text{"OK"}}$

E14-1.7.3 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$H_1 = \gamma_f\ h_{eq}\ (h' - t)\ k_a\ \cos(90\ deg - \theta + \delta)$ $\boxed{H_1 = 1.2}$ kip/ft

$H_2 = \frac{1}{2}\ \gamma_f\ (h' - t)^2\ k_a\ \cos(90\ deg - \theta + \delta)$ $\boxed{H_2 = 5.0}$ kip/ft

$M_1 = H_1\left(\frac{h' - t}{2}\right)$ $\boxed{M_1 = 10.0}$ kip-ft/ft

$M_2 = H_2\left(\frac{h' - t}{3}\right)$ $\boxed{M_2 = 28.4}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$H_{u1} = 1.75\ H_1 + 1.50\ H_2$ $\boxed{H_{u1} = 9.6}$ kip/ft

$M_{u1} = 1.75\ M_1 + 1.50\ M_2$ $\boxed{M_{u1} = 60.0}$ kip-ft/ft

for **Service I**:

$H_{u3} = 1.00\ H_1 + 1.00\ H_2$ $\boxed{H_{u3} = 6.2}$ kip/ft

$M_{u3} = 1.00\ M_1 + 1.00\ M_2$ $\boxed{M_{u3} = 38.4}$ kip-ft/ft

E14-1.7.3.1 Evaluate Stem Shear Strength at Footing

$V_u = H_{u1}$ $\boxed{V_u = 9.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\ \beta\lambda\sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 10.0$ in (bar spacing)

$Bar_{No} = 8$ (transverse bar size)

$Bar_D = 1.00$ in (transverse bar diameter)

$Bar_A = 0.79$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ $\boxed{A_s = 0.95}$ in^2/ft

$$d_s = T_b\,12 - cover - \frac{Bar_D}{2}$$ $\boxed{d_s = 23.0}$ in

$$a = \frac{A_s\,f_y}{\alpha_1\,f'_c\,b}$$ $\boxed{a = 1.6}$ in

$$d_{v1} = d_s - \frac{a}{2}$$ $\boxed{d_{v1} = 22.2}$ in

$$d_{v2} = 0.9\,d_s$$ $\boxed{d_{v2} = 20.7}$ in

$$d_{v3} = 0.72\,T_b\,12$$ $\boxed{d_{v3} = 18.4}$ in

$$d_v = \max(d_{v1}, d_{v2}, d_{v3})$$ $\boxed{d_v = 22.2}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_c = 0.0316\,\beta\,\lambda\sqrt{f'_c}\,b\,d_v$$ $\boxed{V_c = 31.5}$ kip/ft

$$V_{n1} = V_c$$ $\boxed{V_{n1} = 31.5}$ kip/ft

$$V_{n2} = 0.25\,f'_c\,b\,d_v$$ $\boxed{V_{n2} = 233.1}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 31.5}$ kip/ft

$$V_r = \phi_v\,V_n$$ $\boxed{V_r = 28.4}$ kip/ft

$\boxed{V_u = 9.6}$ kip/ft

Is V_u less than V_r? $\boxed{check = "OK"}$

E14-1.7.3.2 Evaluate Stem Flexural Strength at Footing

$M_u = M_{u1}$ $\boxed{M_u = 60.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$$M_n = A_s\, f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$$ $\boxed{M_n = 105.2}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 1.87}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\, M_n$ $\boxed{M_r = 94.7}$ kip-ft/ft

$\boxed{M_u = 60.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\, \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24 \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.45}$ ksi

$$I_g = \frac{1}{12}\, b \left(T_b\, 12\right)^3$$ $\boxed{I_g = 16581}$ in^4

$$y_t = \frac{1}{2}\, T_b\, 12$$ $\boxed{y_t = 12.8}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1301}$ in^3

$M_{cr\_s} = 1.1\ f_r\ S_c\ \frac{1}{12}$ from E14-1.7.1.2 $\boxed{M_{cr\_s} = 53.5}$ kip-ft/ft

$\boxed{1.33\ M_u = 79.9}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33^*M_u? $\boxed{check = "OK"}$

Check the Service Ib crack control requirements in accordance with **LRFD [5.7.3.4]**

$\rho = \frac{A_s}{d_s\ b}$ $\boxed{\rho = 0.00343}$

$n = \frac{E_s}{E_c}$ $\boxed{n = 8.09}$

$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n$ $\boxed{k = 0.210}$

$j = 1 - \frac{k}{3}$ $\boxed{j = 0.930}$

$d_c = cover + \frac{Bar_D}{2}$ $\boxed{d_c = 2.5}$ in

$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12 \leq 0.6\ f_y$ $\boxed{f_{ss} = 22.7}$ ksi $\leq 0.6\ f_y$ O.K.

$h = T_b\ 12$

$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$ $\boxed{\beta_s = 1.2}$

$\gamma_e = 1.0$ for Class 1 exposure

$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c$ $\boxed{s_{max} = 21.7}$ in

$\boxed{s = 10.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = "OK"}$

E14-1.7.3.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-1.7.4 Temperature and Shrinkage Steel

Look at temperature and shrinkage requirements

E14-1.7.4.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required. However, #4 bars at 18" o.c. (max) are placed longitudinally to serve as spacers.

E14-1.7.4.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with **LRFD [5.10.8** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_D = 0.50$ in (temperature and shrinkage bar diameter)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 216.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right) f_y}$$ Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \leq A_s \leq 0.60$? $\boxed{check = "OK"}$

Is $A_s > A_{ts}$? $\boxed{check = "OK"}$

Check the maximum spacing requirements

$s_1 = \min(3\,h_s, 18)$ $\boxed{s_1 = 18.0}$ in

$s_2 = \begin{cases} 12 & \text{if } h_s > 18 \\ s_1 & \text{otherwise} \end{cases}$ For walls and footings (in) $\boxed{s_2 = 18.0}$ in

$s_{max} = \min(s_1, s_2)$ $\boxed{s_{max} = 18.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-1.8 Summary of Results

List all summaries.

E14-1.8.1 Summary of External Stability

Based on the defined project parameters the following external stability checks have been satisfied:

External Check	CDR Strength
Sliding	1.29
Eccentricity	1.94
Bearing	1.64

Table E14-1.8-1
Summary of External Stability Computations

E14-1.8.2 Summary of Wall Strength Design

The required wall reinforcing from the previous computations are presented in Figure E14-1.9-1.

E14-1.8.3 Drainage Design

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are met by providing granular, free draining backfill material with a pipe underdrain located at the bottom of the wall (Assumed wall is adjacent to sidewalk) as shown in Figure E14-1.9-1.

E14-1.9 Final CIP Concrete Wall Schematic

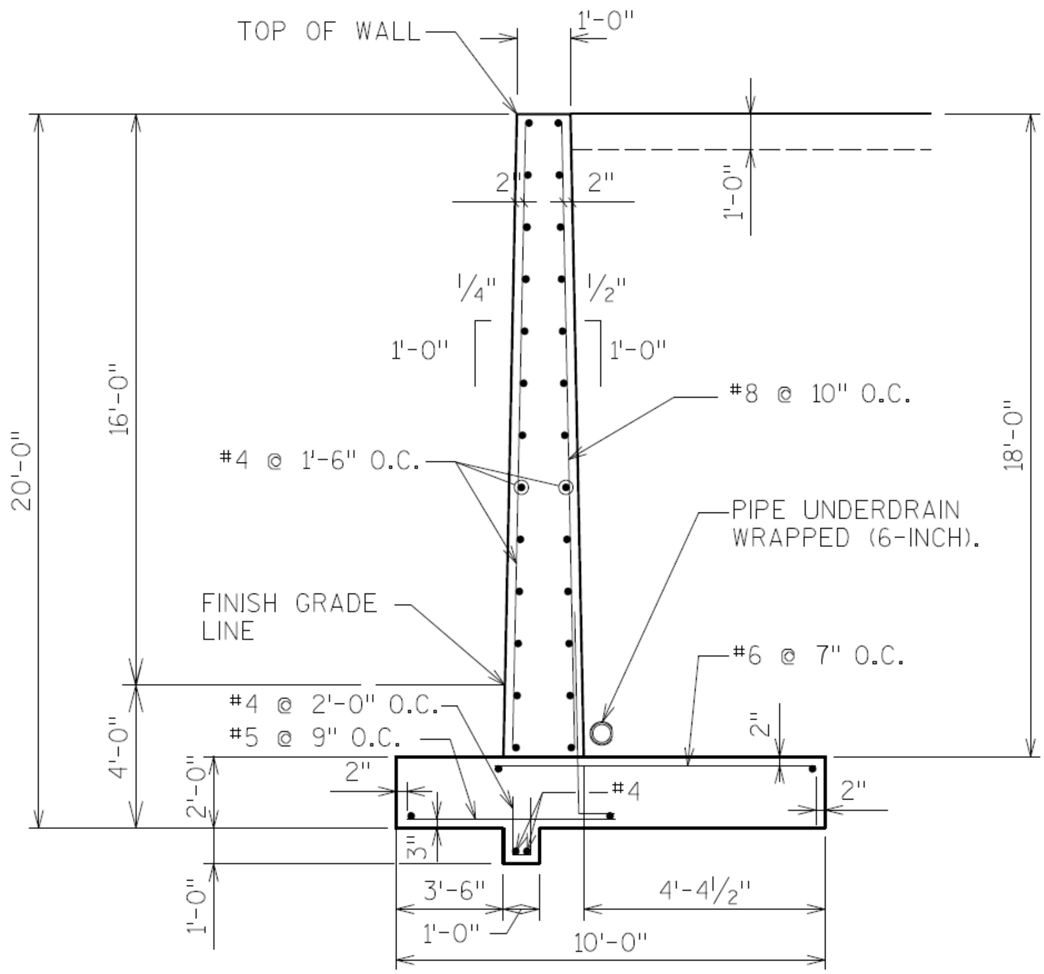

Figure E14-1.9-1
Cast-In-Place Wall Schematic

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E14-2 Precast Panel Steel Reinforced MSE Wall, LRFD

General

This example shows design calculations for MSE wall with precast concrete panel facings conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for external stability (sliding, eccentricity and bearing) and internal stability (soil reinforcement stress and pullout) will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.6.3.3 are used for the wall design.

E14-2.1 Establish Project Requirements

The following MSE wall shall have compacted freely draining soil in the reinforced zone and will be reinforced with metallic (inextensible) strips as shown in Figure E14-2.1-1. External stability is the designer's (WisDOT/Consultant) responsibility and internal stability and structural components are the contractors responsibility.

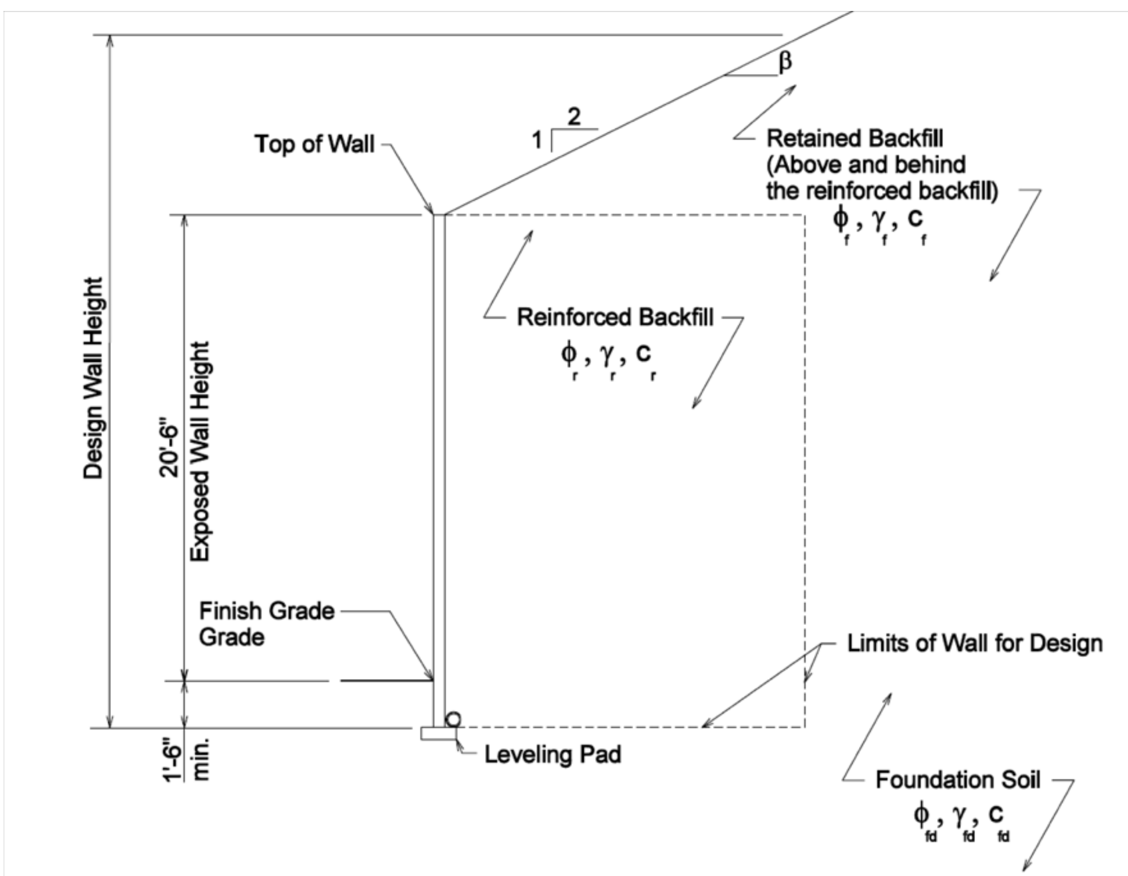

Figure E14-2.1-1
MSE Wall with Sloping Backfill

Wall Geometry

$H_e = 20.5$ Exposed wall height, ft

$H = H_e + 1.5$ Design wall height, ft (assume 1.5 ft wall embedment)

$\theta = 90\ \text{deg}$ Angle of back face of wall to horizontal

$\beta = 26.565\ \text{deg}$ Inclination of ground slope behind face of wall (2H:1V)

E14-2.2 Design Parameters

Project Parameters

$Design\_Life = 75$ Wall design life, years (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Reinforced Backfill Soil Design Parameters

$\phi_r = 30\ \text{deg}$ Angle of internal friction **LRFD [11.10.5.1]**

$\gamma_r = 0.120$ Unit of weight, kcf

$c_r = 0$ Cohesion, psf

Retained Backfill Soil Design Parameters

$\phi_f = 29\ \text{deg}$ Angle of internal friction

$\gamma_f = 0.120$ Unit of weight, kcf

$c_f = 0$ Cohesion, psf

Foundation Soil Design Parameters

$\phi_{fd} = 31\text{deg}$ Angle of internal friction

$\gamma_{fd} = 0.125$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, psf

Factored Bearing Resistance of Foundation Soil

$q_R = 10.0$ Factored resistance at the strength limit state, ksf

Note: The factored bearings resistance, q_R, was assumed to be given in the Site Investigation Report. If not provided q_R shall be determined by calculating the nominal bearing resistance, q_n, per **LRFD [Eq 10.6.3.1.2a-1]** and factored with the bearing resistance factor, ϕ_b, for MSE walls (i.e., $q_R = \phi_b q_n$).

Precast Concrete Panel Facing Parameters

$S_{vt} = 2.5$ Vertical spacing of reinforcement, ft

Note: vertical spacing should not exceed 2.7 ft without full scale test data **LRFD [11.10.6.2.1]**

$w_p = 5.0$ Width of precast concrete panel facing, ft

$h_p = 5.0$ Height of precast concrete panel facing, ft

$t_p = 6.0$ Thickness of precast concrete panel facing, in

Soil Reinforcement Design Parameters

Galvanized steel ribbed strips Reinforcing type

$F_y = 65$ Reinforcing strip yield strength, ksi (Grade 65)

$b\_mm = 50$ Reinforcing strip width, mm

$$b = \frac{b\_mm}{25.4} \qquad \boxed{b = 1.97} \text{ in}$$

$E_{n\_mm} = 4$ Reinforcing strip thickness, mm

$$E_n = \frac{E_{n\_mm}}{25.4} \qquad \boxed{E_n = 0.16} \text{ in}$$

$Zinc = 3.4$ Zinc coating, mils (Minimum **LRFD [11.10.6.4.2a]**)

Live Load Surcharge Parameters

$SUR = 0.100$ Live load surcharge for walls without traffic, ksf (14.4.5.4.2)

Resistance Factors

$\phi_s = 1.00$	Sliding of MSE wall at foundation **LRFD [Table 11.5.7-1]**
$\phi_b = 0.65$	Bearing resistance **LRFD [Table 11.5.7-1]**
$\phi_t = 0.75$	Tensile resistance (steel strips) **LRFD [Table 11.5.7-1]**
$\phi_p = 0.90$	Pullout resistance **LRFD [Table 11.5.7-1]**

E14-2.3 Estimate Depth of Embedment and Length of Reinforcement

For this example it is assumed that global stability does not govern the required length of soil reinforcement.

Embedment Depth, d_e

Frost-susceptible material is assumed to be not present or that it has been removed and replaced with nonfrost susceptible material per **LRFD [11.10.2.2]**. There is also no potential for scour. Therefore, the minimum embedment, d_e, shall be the greater of 1.5 ft (14.6.4) or H/20 **LRFD [Table C11.10.2.2-1]**

Note: While AASHTO allows the d_e value of 1.0 ft on level ground, the embedment depth is limited to 1.5 ft by WisDOT policy as stated in Chapter 14.

$$\frac{H}{20} = 1.1 \quad \text{ft}$$

$$d_e = \max\left(\frac{H}{20}, 1.5\right) \qquad \boxed{d_e = 1.50} \text{ ft}$$

Therefore, the initial design wall height assumption was correct.

$$H_e = 20.5 \quad \text{ft}$$

$$H = H_e + 1.5 \qquad \boxed{H = 22.00} \text{ ft}$$

Length of Reinforcement, L

In accordance with **LRFD [11.10.2.1]** the minimum required length of soil reinforcement shall be the greater of 8 feet or 0.7H. Due to the sloping backfill surcharge and live load surcharge a longer reinforcement length of 0.9H will be used in this example. The length of reinforcement will be uniform throughout the entire wall height.

$0.9\,H = 19.8$ ft

$L_{user} = 20.0$ ft

$L = \max(8.0, 0.9\,H, L_{user})$ $\boxed{L = 20.00}$ ft

Height of retained fill at the back of the reinforced soil, h

$h = H + L\tan(\beta)$ $\boxed{h = 32.00}$ ft

E14-2.4 Permanent and Transient Loads

In this example, load types EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used as shown in Figure E14-2.4-1. Due to the relatively thin wall thickness the weight and width of the concrete facing will be ignored. Passive soil resistance will also be ignored.

E14-2.4.1 Compute Active Earth Pressure

Compute the coefficient of active earth pressure (k_a) using Coulomb Theory **LRFD [Eq 3.11.5.3-1]** with the wall backfill material interface friction angle, δ, set equal to β (i.e. $\delta=\beta$) **LRFD [11.10.5.2].** The retained backfill soil will be used (i.e., $k_a=k_{af}$)

$\phi_f = 29$ deg

$\beta = 26.565$ deg

$\theta = 90$ deg

$\delta = \beta$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\,\sin(\phi_f - \beta)}{\sin(\theta - \delta)\,\sin(\theta + \beta)}}\right)^2 \qquad \boxed{\Gamma = 1.462}$$

$$k_{af} = \frac{\sin(\theta + \phi_f)^2}{\Gamma\,\sin(\theta)^2\,\sin(\theta - \delta)} \qquad \boxed{k_{af} = 0.585}$$

E14-2.4.2 Compute Unfactored Loads

The forces and moments are computed using Figure E14-2.4-1 by their appropriate LRFD load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**

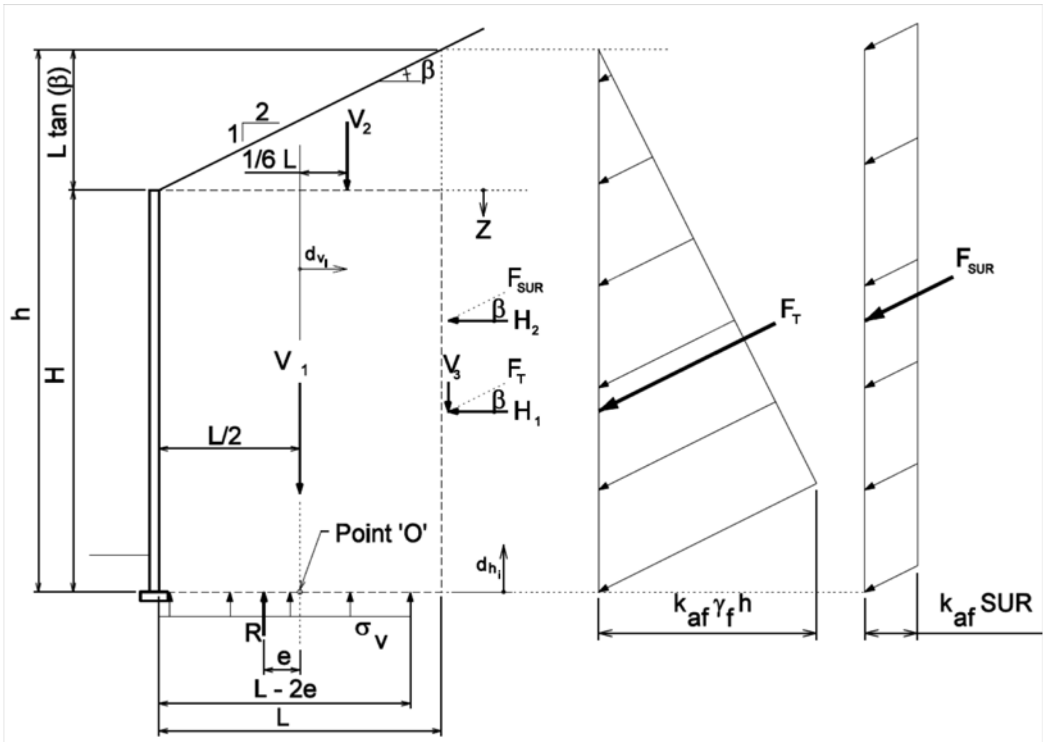

Figure E14-2.4-1
MSE Wall - External Stability

Active Earth Force Resultant, (kip/ft), F_T

$$F_T = \frac{1}{2} \gamma_f h^2 k_{af}$$ Active earth force resultant (EH) $\boxed{F_T = 35.9}$

Live Load Surcharge Resultant, (kip/ft), F_{SUR}

$$F_{SUR} = SUR\ h\ k_{af}$$ Live load surcharge (LS) $\boxed{F_{SUR} = 1.9}$

Vertical Loads, (kip/ft), V_i

$$V_1 = \gamma_r\ H\ L$$ Soil backfill - reinforced soil (EV) $\boxed{V_1 = 52.8}$

$$V_2 = \frac{1}{2} \gamma_f\ L\ (L\ \tan(\beta))$$ Soil backfill - backslope (EV) $\boxed{V_2 = 12.0}$

$$V_3 = F_T \sin(\beta)$$ Active earth force resultant (vertical component - EH) $\boxed{V_3 = 16.1}$

Moments produced from vertical loads about Point 'O', (kip-ft/ft) MV_i

Moment Arm		Moment	
$d_{v1} = 0$	$\boxed{d_{v1} = 0.0}$	$MV_1 = V_1\, d_{v1}$	$\boxed{MV_1 = 0.0}$
$d_{v2} = \frac{1}{6}L$	$\boxed{d_{v2} = 3.3}$	$MV_2 = V_2\, d_{v2}$	$\boxed{MV_2 = 40.0}$
$d_{v3} = \frac{L}{2}$	$\boxed{d_{v3} = 10.0}$	$MV_3 = V_3\, d_{v3}$	$\boxed{MV_3 = 160.7}$

Horizontal Loads, (kip/ft), H_i

$H_1 = F_T \cos(\beta)$	Active earth force resultant (horizontal component - EH)	$\boxed{H_1 = 32.1}$
$H_2 = F_{SUR} \cos(\beta)$	Live load surcharge resultant (horizontal component - LS)	$\boxed{H_2 = 1.7}$

Moments produced from horizontal loads about Point 'O', (kip-ft/ft), MH_i

Moment Arm		Moment	
$d_{h1} = \frac{h}{3}$	$\boxed{d_{h1} = 10.7}$	$MH_1 = H_1\, d_{h1}$	$\boxed{MH_1 = 342.8}$
$d_{h2} = \frac{h}{2}$	$\boxed{d_{h2} = 16.0}$	$MH_2 = H_2\, d_{h2}$	$\boxed{MH_2 = 26.8}$

Summary of Unfactored Forces & Moments:

Load			**Moment Arm**		**Moment**		**LRFD Load Type**
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Soil backfill	52.80	d_{v1}	0.0	MV_1	0.0	EV
V_2	Soil backfill	12.00	d_{v2}	3.3	MV_2	40.0	EV
V_3	Active earth pressure	16.10	d_{v3}	10.0	MV_3	160.7	EH

Table E14-2.4-1
Unfactored Vertical Forces & Moments

Load			**Moment Arm**		**Moment**		**LRFD Load Type**
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Active earth pressure	32.1	d_{h1}	10.7	MH_1	342.8	EH
H_2	Live load surcharge	1.70	d_{h2}	16.0	MH_2	26.8	LS

Table E14-2.4-2
Unfactored Horizontal Forces & Moments

E14-2.4.3 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all load modifiers to one ($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be checked in this example:

Load Combination Limit State	EV	LS	EH
Strength Ia (minimum)	$\gamma_{EVmin} = 1.00$	$\gamma_{LSmin} = 1.75$	$\gamma_{EHmin} = 0.90$
Strength Ib (maximum)	$\gamma_{EVmax} = 1.35$	$\gamma_{LSmax} = 1.75$	$\gamma_{EHmax} = 1.50$
Service I (max/min)	$\gamma_{EV} = 1.00$	$\gamma_{LS} = 1.00$	$\gamma_{EH} = 1.00$

Load Combination	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.35	1.75	1.75	1.50	Bearing, T_{max}
Service I	1.00	1.00	1.00	1.00	Pullout (σ_v)

Table E14-2.4-3
Unfactored Horizontal Forces & Moments

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)}$ = 0.9, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_3\gamma_{EH(max)}$ and $H_1\gamma_{EH(max)}$ or $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(min)}$, not $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(max)}$.
- T_{max1} (Pullout) is calculated without live load and T_{max2} (Rupture) is calculated with live load.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-2.4.3 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$V_{EV} = V_1 + V_2$ $\boxed{V_{EV} = 64.8}$

$V_{EH} = V_3$ $\boxed{V_{EH} = 16.1}$

$H_{EH} = H_1$ $\boxed{H_{EH} = 32.1}$

$H_{LS} = H_2$ $\boxed{H_{LS} = 1.7}$

Unfactored moments by load type (kip-ft/ft)

$M_{EV} = MV_1 + MV_2$ $\boxed{M_{EV} = 40.0}$

$M_{EH1} = MV_3$ $\boxed{M_{EH1} = 160.7}$

$M_{EH2} = MH_1$ $\boxed{M_{EH2} = 342.8}$

$M_{LS2} = MH_2$ $\boxed{M_{LS2} = 26.8}$

Factored vertical loads by limit state (kip/ft)

$V\_Ia = n(1.00V_{EV} + 1.50\ V_{EH})$ $\boxed{V\_Ia = 88.9}$

$V\_Ib = n(1.35V_{EV} + 1.50\ V_{EH})$ $\boxed{V\_Ib = 111.6}$

$V\_Ser = n(1.00V_{EV} + 1.00\ V_{EH})$ $\boxed{V\_Ser = 80.9}$

Factored horizontal loads by limit state (kip/ft)

$H\_Ia = n(1.75H_{LS} + 1.50H_{EH})$ $\boxed{H\_Ia = 51.1}$

$H\_Ib = n(1.75H_{LS} + 1.50H_{EH})$ $\boxed{H\_Ib = 51.1}$

$H\_Ser = n(1.00H_{LS} + 1.00H_{EH})$ $\boxed{H\_Ser = 33.8}$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$MV\_Ia = n(1.00M_{EV} + 1.50\ M_{EH1})$ $\boxed{MV\_Ia = 281.0}$

$MV\_Ib = n(1.35M_{EV} + 1.50\ M_{EH1})$ $\boxed{MV\_Ib = 295.0}$

$MV\_Ser = n(1.00M_{EV} + 1.00\ M_{EH1})$ $\boxed{MV\_Ser = 200.7}$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$MH\_Ia = n(1.75M_{LS2} + 1.50\ M_{EH2})$ $\boxed{MH\_Ia = 561.1}$

$MH\_Ib = n(1.75M_{LS2} + 1.50\ M_{EH2})$ $\boxed{MH\_Ib = 561.1}$

$MH\_Ser = n(1.00M_{LS2} + 1.00\ M_{EH2})$ $\boxed{MH\_Ser = 369.6}$

Summary of Factored Forces & Moments:

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	88.9	281.0	51.1	561.1
Strength Ib	111.6	295.0	51.1	561.1
Service I	80.9	200.7	33.8	369.6

Table E14-2.4-4
Summary of Factored Loads & Moments

E14-2.5 Evaluate External Stability of MSE Wall

Three potential external failure mechanisms will be considered in this example (sliding at the base, limiting eccentricity and bearing resistance). Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-2.5.1 Sliding Resistance at Base of MSE Wall

The following calculations are based on **Strength Ia**:

Factored Sliding Force

$R_u = H\_Ia$ $\boxed{R_u = 51.14}$ kip/ft

Sliding Resistance

To compute the coefficient of sliding friction for discontinuous reinforcement use the lesser friction angle of the reinforced back fill, ϕ_r , or foundation soil, ϕ_{fd}, **LRFD [11.10.5.3]**.

$\phi_\mu = \min(\phi_r, \phi_{fd})$ $\boxed{\phi_\mu = 30 \text{ deg}}$

$\mu = \tan(\phi_\mu)$ $\boxed{\mu = 0.577}$

$V\_Ia = 88.9$ Factored vertical load, kip/ft

$V_{Nm} = \mu\ V\_Ia$ $\boxed{V_{Nm} = 51.3}$ kip/ft

$\phi_s = 1.0$

$R_R = \phi_s\ V_{Nm}$ $\boxed{R_R = 51.33}$ kip/ft

<u>Capacity:Demand Ratio (CDR)</u>

$CDR_{Sliding} = \dfrac{R_R}{R_u}$ $\boxed{CDR_{Sliding} = 1.00}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-2.5.2 Limiting Eccentricity at Base of MSE Wall

The location of the resultant of the reaction forces is limited to the middle two-thirds of the base width for a soil foundation (i.e., e_{max} = L/3) **LRFD [11.6.3.3]**. The following calculations are based on **Strength Ia**:

Maximum eccentricity

$$e_{max} = \frac{L}{3}$$

$\boxed{e_{max} = 6.67}$ ft

Compute wall eccentricity (distance from Point 'O' in Figure E14-2.4-1)

$\Sigma M_R = MV\_Ia$ Summation of resisting moments for Strength Ia

$\Sigma M_O = MH\_Ia$ Summation of overturning moments for Strength Ia

$\Sigma V = V\_Ia$ Summation of vertical loads for Strength Ia

$\boxed{\Sigma M_R = 281.0}$ kip-ft/ft

$\boxed{\Sigma M_O = 561.1}$ kip-ft/ft

$\boxed{\Sigma V = 88.9}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$

$\boxed{e = 3.15}$ ft

Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} = \frac{e_{max}}{e}$$

$\boxed{CDR_{Eccentricity} = 2.12}$

Is the CDR $\geq$ 1.0 ?

$\boxed{check = \text{"OK"}}$

E14-2.5.3 Bearing Resistance at base of MSE Wall

The following calculations are based on **Strength Ib**:

Compute wall eccentricity (distance from Point 'O' in Figure E14-2.4-1)

$\Sigma M_R = MV\_Ib$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ Summation of vertical loads for Strength Ib

$\boxed{\Sigma M_R = 295.0}$ kip-ft/ft

$\boxed{\Sigma M_O = 561.1}$ kip-ft/ft

$\boxed{\Sigma V = 111.6}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$

$\boxed{e = 2.38}$ ft

Compute the ultimate bearing stress

$\sigma_{\cdot V}$ = Ultimate bearing stress
L = Bearing length
e = Eccentricity (resultant produced by extreme bearing resistance loading)

Note: For the bearing resistance calculations the effective bearing width, B' = L-2e, is used instead of the actual width. Also, when the eccentricity, e, is negative: B'=L. The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**.

$$\sigma_V = \frac{\Sigma V}{L - 2\,e}$$

$\boxed{\sigma_V = 7.33}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 10.00}$ ksf/ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Bearing} = \frac{q_R}{\sigma_V}$$

$\boxed{CDR_{Bearing} = 1.37}$

Is the CDR $\geq$ 1.0 ?

$\boxed{check = \text{"OK"}}$

E14-2.6 Evaluate Internal Stability of MSE Wall

Note: MSE walls are a proprietary wall system and the internal stability computations shall be performed by the wall supplier.

Internal stability shall be checked for 1) pullout and 2) rupture in accordance with **LRFD [11.10.6]**. The factored tensile load, T_{max}, is calculated twice for internal stability checks for vertical stress (σ_V) calculations. For pullout T_{max1} is determined by excluding live load surcharge. For rupture T_{max2} is determined by including live load surcharge. In this example, the maximum reinforcement loads are calculated using the Simplified Method.

The location of the potential failure surface for a MSE wall with metallic strip or grid reinforcements (inextensible) is shown in Figure E14-2.6-1.

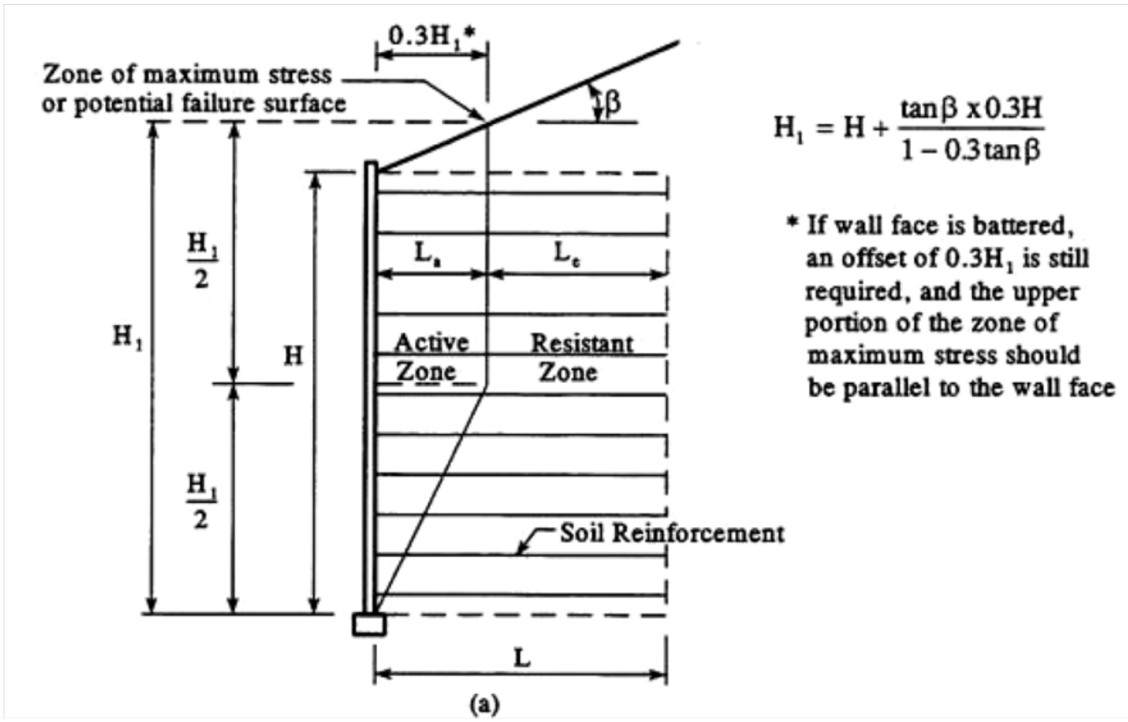

Figure E14-2.6-1
MSE Wall - Internal Stability (Inextensible Reinforcement)
FHWA [Figure 4-9]

E14-2.6.1 Establish the Vertical Layout of Soil Reinforcement

Soil reinforcement layout is shown in Table E14-2.6-1. The location of the reinforcement levels corresponds to the vertical depth, Z, into the reinforced soil. The vertical layout was determined by the industry practice of 2.5 ft vertical spacing for steel ribbed strip reinforcement. The top level vertical spacing was adjusted to fit the height of the wall. Computations for determining the maximum tension, T_{max}, at Z = 8.25 ft are as follows:

$Layer = 4$ Layer of reinforcement (from top)

$Z = 8.25$ Depth below top of wall, ft

$S_{vt} = 2.5$ Vertical spacing of reinforcement, ft

$w_p = 5.00$ Width of precast concrete panel facing, ft

Calculate the upper and lower tributary depths based on the reinforcement vertical spacing

$$Z_{neg} = Z - \frac{S_{vt}}{2}$$ $\boxed{Z_{neg} = 7.0}$ ft

$$Z_{pos} = Z + \frac{S_{vt}}{2}$$ $\boxed{Z_{pos} = 9.5}$ ft

Layer	Z (ft)	Z (ft)	Z^+ (ft)	S_{vt} (ft)
1	0.75	0	0.75+0.5(3.25-0.75)= 2.0	2.00
2	3.25	3.25-0.5(3.25-0.75)= 2.0	3.25+0.5(5.75-3.25)= 4.5	2.50
3	5.75	5.75-0.5(5.75-3.25)= 4.5	5.75+0.5(8.25-5.75)= 7.0	2.50
4	**8.25**	**8.25-0.5(8.25-5.75)= 7.0**	**8.25+0.5(10.75-8.25)= 9.5**	**2.50**
5	10.75	10.75-0.5(10.75-8.25)= 9.5	10.75+0.5(13.25-10.75)= 12.0	2.50
6	13.25	13.25-0.5(13.25-10.75)= 12.0	13.25+0.5(15.75-13.25)= 14.5	2.50
7	15.75	15.75-0.5(15.75-13.25)= 14.5	15.75+0.5(18.25-15.75)= 17.0	2.50
8	18.25	18.25-0.5(18.25-15.75)= 17.0	18.25+0.5(20.75-18.25)= 19.5	2.50
9	20.75	20.75-0.5(20.75-18.25)= 19.5	22	2.50

Table E14-2.6-1
Summary of Computations for Reinforcement Spacing, S_{vt}

E14-2.6.2 Compute Horizontal Stress and Maximum Tension, T_{max}

Factored horizontal stress

$$\sigma_H = \gamma_P (\sigma_v k_r + \Delta\sigma_H)$$ **LRFD [Equation 11.10.6.2.1-1]**

γ_p = Load factor for vertical earth pressure (γ_{EVmax})
k_r = Horizontal pressure coefficient
σ_v = Pressure due to gravity and surcharge for pullout, T_{max1} ($\gamma_r Z_{trib}+\sigma_2$)
σ_v = Pressure due to gravity and surcharge for pullout resistance ($\gamma_r Z_{p\text{-}PO}$)
σ_v = Pressure due to gravity and surcharge for rupture, T_{max2} ($\gamma_r Z_{trib} + \sigma_2 + q$)
$\Delta\sigma_H$ = Horizontal pressure due to concentrated horizontal surcharge load
Z = Reinforcement depth for max stress Figure E14-2.6-2
Z_p = Depth of soil at reinforcement layer potential failure plane
$Z_{p\text{-}ave}$ = Average depth of soil at reinforcement layer in the effective zone
σ_2 = Equivalent uniform stress from backslope $(0.5(0.7)L\tan\beta)\gamma_f$
q = Surcharge load ($q = SUR$), ksf

To compute the lateral earth pressure coefficient, k_r , a k_a multiplier is used to determine k_r for each of the respective vertical tributary spacing depths (Z_{pos}, Z_{neg}). The k_a multiplier is determined using Figure E14-2.6-2. To calculate k_a it is assumed that $\delta=\beta$ and $\beta = 0$; thus, $k_a=\tan^2(45-\phi_f/2)$ **LRFD [Equation C11.10.6.2.1-1]**

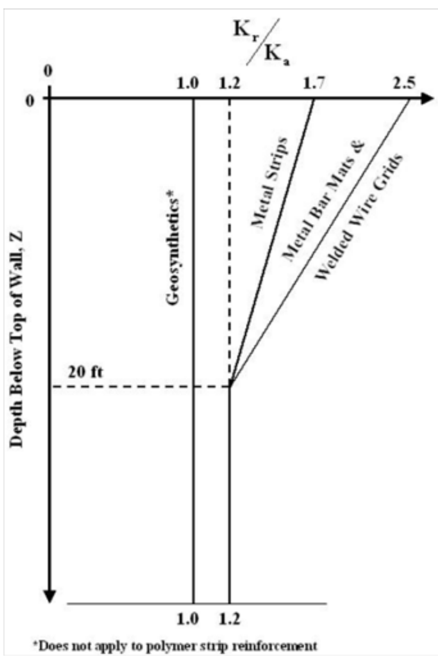

Figure E14-2.6-2
k_r/k_a Variation with MSE Wall Depth
FHWA [Figure 4-10]

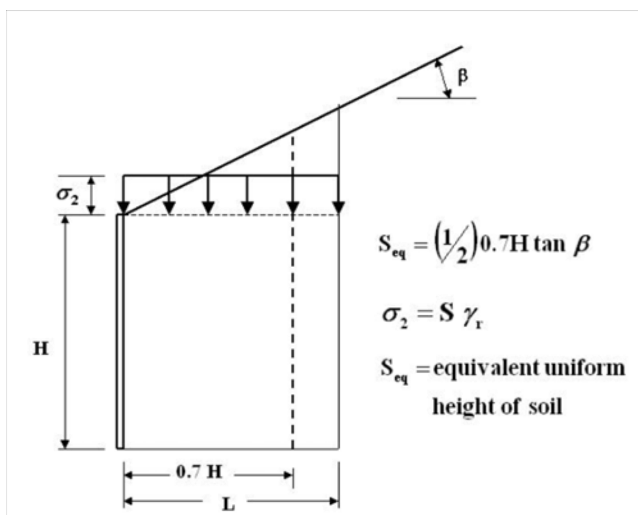

Figure E14-2.6-3
Calculation of Vertical Stress
FHWA [Figure 4-11]

Calculate the coefficient of active earth pressure, k_a

$\phi_f = 29\ deg$

$\boxed{k_a = 0.347}$

$$k_a = \tan\left(45\ deg - \frac{\phi_f}{2}\right)^2$$

Compute the internal lateral earth pressure coefficient limits based on applying a k_a multiplier as shown in Figure E14-2.6-2. For inextensible steel ribbed strips the k_a multiplier decreases linearly from the top of the reinforced soil zone to a depth of 20 ft. Thus, the k_a multiplier will vary from 1.7 at Z=0 ft to 1.2 at Z=20 ft. To compute k_r apply these values to the coefficient of active earth pressure.

$k_{r\_0ft} = 1.7\ k_a$ $\boxed{k_{r\_0ft} = 0.590}$

$k_{r\_20ft} = 1.2\ k_a$ $\boxed{k_{r\_20ft} = 0.416}$

Compute the internal lateral earth pressure coefficients, k_r, for each of the respective tributary depths. Since both depths, Z_{neg} and Z_{pos}, are less than 20 ft k_r will be interpolated at their respective depths

$$k_{r\_neg} = k_{r\_20ft} + \frac{(20 - Z_{neg})(k_{r\_0ft} - k_{r\_20ft})}{20}$$ $\boxed{k_{r\_neg} = 0.529}$

$$k_{r\_pos} = k_{r\_20ft} + \frac{(20 - Z_{pos})(k_{r\_0ft} - k_{r\_20ft})}{20}$$ $\boxed{k_{r\_pos} = 0.507}$

Compute effective (resisting) length, L_e

$Z = 8.25$ ft Refer to Figure E14-2.6-1. ($\Delta H = H_1 - H$)

$H = 22.0$ ft

$L = 20$ ft

$$\Delta H = \frac{\tan(\beta)\,(0.3\ H)}{1 - 0.3\tan(\beta)}$$ $\boxed{\Delta H = 3.88}$ ft

$H_1 = H + \Delta H$ $\boxed{H_1 = 25.9}$ ft

$$L_a = \begin{cases} 0.3\ H_1 & \text{if } Z \le \frac{H_1}{2} - \Delta H \\ \frac{H - Z}{\frac{H_1}{2}}(0.3\ H_1) & \text{otherwise} \end{cases}$$ $\boxed{L_a = 7.76}$ ft

$L_e = \max(L - L_a, 3)$ $\boxed{L_e = 12.24}$ ft

Note: L_e shall be greater than or equal to 3 feet **LRFD [11.10.6.3.2]**

E14-2.6.3 Establish Nominal and Factored Pullout Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H, at Z by averaging the upper and lower tributary values (Z_{neg} and Z_{pos}). Since there is no horizontal stresses from concentrated dead loads values $\Delta\sigma_H$ is set to zero.

$$\sigma_H = \gamma_{EVmax} (\gamma_r Z_{trib}+\sigma_2)k_r$$

Surcharge loads

$$\sigma_2 = \frac{1}{2} 0.7\ H \tan(\beta)\ \gamma_f$$ $\boxed{\sigma_2 = 0.46}$ ksf/ft

Horizontal stress at Z_{neg} and Z_{pos}

$$\sigma_{H\_neg} = \gamma_{EVmax} \left(\gamma_r Z_{neg} + \sigma_2\right) k_{r\_neg}$$ $\boxed{\sigma_{H\_neg} = 0.93}$ ksf/ft

$$\sigma_{H\_pos} = \gamma_{EVmax} \left(\gamma_r Z_{pos} + \sigma_2\right) k_{r\_pos}$$ $\boxed{\sigma_{H\_pos} = 1.10}$ ksf/ft

Horizontal stress at Z

$$\sigma_H = 0.5\left(\sigma_{H\_pos} + \sigma_{H\_neg}\right)$$ $\boxed{\sigma_H = 1.01}$ ksf/ft

Compute the maximum tension, T_{max1} , at Z

$$A_{trib} = S_{vt}\ w_p$$ $\boxed{A_{trib} = 12.50}$ ft^2

$$T_{max1} = \sigma_H\ A_{trib}$$ $\boxed{T_{max1} = 12.67}$ kip/strip

Compute effective vertical stress for pullout resistance, σ_v

$$Z_{p\_PO} = Z + 0.5 \tan(\beta) \left(L_a + L\right)$$ $\boxed{Z_{p\_PO} = 15.2}$ ft

$\gamma_{EV} = 1.00$ Unfactored vertical stress for pullout resistance **LRFD [11.10.6.3.2]**

$$\sigma_v = \gamma_{EV}\ \gamma_r\ Z_{p\_PO}$$ $\boxed{\sigma_v = 1.82}$ ksf

Compute pullout resistance factor, F*

The coefficient of uniformity, C_u, shall be computed based on backfill gradations D_{60}/D_{10}. If the backfill material is unknown at the time of design a conservative assumption of C_u=4 should be assumed **LRFD [11.10.6.3.2]**.

The pullout resistance factor, F*, for inextensible steel ribbed strips decreases linearly from the top of the intersection of the failure plane with the top of the reinforced soil zone. Thus, F* will vary from 1.2+log C_u (<= 2.0) at Z=0 ft to $\tan(\phi_r)$ at Z=20 ft. Since no product-specific pullout test data is provided at the time of design the default value for F* will be used as provided by **LRFD [Figure 11.10.6.3.2-1]**.

$C_u = 4$ Coefficient of uniformity (C_u=4 default value) **LRFD [11.10.6.3.2]**

$F'_{0ft} = \min(2.00, 1.2 + \log(C_u))$ $\boxed{F'_{0ft} = 1.80}$

$F'_{20ft} = \tan(\phi_r)$ $\boxed{F'_{20ft} = 0.58}$

$$F' = \begin{cases} F'_{20ft} + \dfrac{20.0 - Z}{20}\left(F'_{0ft} - F'_{20ft}\right) & \text{if } Z \le 20.0 \\ \tan(\phi_r) & \text{otherwise} \end{cases}$$

$\boxed{F' = 1.30}$

Compute nominal pullout resistance, P_r

$\alpha = 1.0$ Scale effect correction factor (steel reinforcement $\alpha = 1.0$ default value) **LRFD [Table 11.10.6.3.2-1]**

$C = 2$ Overall reinforcement surface area geometry factor (strip reinforcement $C = 2.0$) **LRFD [11.10.6.3.2]**

$R_c = 1$ Reinforcement coverage ratio (continuous reinforcement $R_c = 1.0$) **LRFD [11.10.6.4]**

Note: Using strips are considered discontinuous, however the nominal pullout resistance is based on the actual strip width, rather than a unit width, the reinforcement coverage ratio is 1.

$P_r = F' \alpha \sigma_v C R_c L_e b \dfrac{1}{12}$ $\boxed{P_r = 9.49}$ kip/strip

Compute factored pullout resistance, P_{rr}

$\phi_p = 0.9$

$P_{rr} = \phi_p P_r$ $\boxed{P_{rr} = 8.54}$ kip/strip

Determine number of soil reinforcing strips based on pullout resistance, N_p

$N_p = \dfrac{T_{max1}}{P_{rr}}$ $\boxed{N_p = 1.48}$ strips

E14-2.6.4 Establish Nominal and Factored Long-term Tensile Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H

$$\sigma_H = \gamma_{EVmax}\,(\gamma_r Z + \sigma_2 + q)k_r$$

Surcharge loads

$\boxed{\sigma_2 = 0.46}$ ksf/ft

$\boxed{q = 0.10}$ ksf/ft

Horizontal stress at Z_{neg} and Z_{pos}

$\sigma_{H\_neg} = \gamma_{EVmax}\left(\gamma_r Z_{neg} + \sigma_2 + q\right) k_{r\_neg}$ $\boxed{\sigma_{H\_neg} = 1.00}$ ksf/ft

$\sigma_{H\_pos} = \gamma_{EVmax}\left(\gamma_r Z_{pos} + \sigma_2 + q\right) k_{r\_pos}$ $\boxed{\sigma_{H\_pos} = 1.17}$ ksf/ft

Horizontal stress at Z

$\sigma_H = 0.5\left(\sigma_{H\_pos} + \sigma_{H\_neg}\right)$ $\boxed{\sigma_H = 1.08}$ ksf/ft

Compute the maximum tension, T_{max}, at Z

$A_{trib} = S_{vt}\, w_p$ $\boxed{A_{trib} = 12.50}$ ft^2

$T_{max2} = \sigma_H\, A_{trib}$ $\boxed{T_{max2} = 13.55}$ kip/strip

E_c = thickness of metal reinforcement at end of service life (mil)
E_n = nominal thickness of steel reinforcement at construction (mil)
E_s = sacrificial thickness of metal lost by corrosion during service life of structure (mil)
b = width of metal reinforcement

$F_y = 65$ Reinforcing strip yield strength, ksi

$\phi_t = 0.75$ Tensile resistance (steel strip)

$E_n = 0.16$ Reinforcing strip thickness, in

$b = 1.97$ Reinforcing strip width, in

$Zinc = 3.4$ Galvanized coating, mils

Compute the design cross-sectional area of the reinforcement after sacrificial thicknesses have been accounted for during the wall design life per **LRFD [11.10.6.4.2a]**. The zinc coating life shall be calculated based on 0.58 mil/yr loss for the first 2 years and 0.16 mil/yr thereafter. After the depletion of the zinc coating, the steel design life is calculated and used to determine the sacrificial steel thickness after the steel design life. The sacrificial thickness of steel is based on 0.47 mil/yr/side loss.

Design_Life = Coating_Life + Steel_Design_Life = 75 years

$$Coating\_Life = 2 + \frac{Zinc - 2\ 0.58}{0.16} \qquad \boxed{Coating\_Life = 16.0} \text{ years}$$

$$Steel\_Design\_Life = Design\_Life - Coating\_Life \qquad \boxed{Steel\_Design\_Life = 59} \text{ years}$$

$$E_s = \left(\frac{0.47}{1000}\right) Steel\_Design\_Life\ (2) \qquad \boxed{E_s = 0.055} \text{ in}$$

$$E_c = E_n - E_s \qquad \boxed{E_c = 0.102} \text{ in}$$

$$Design\_Strip\_Area = E_c\ b \qquad \boxed{Design\_Strip\_Area = 0.201} \text{ in}^2$$

Compute the Factored Tensile Resistance, T_r

$$T_n = F_y\ Design\_Strip\_Area \qquad \boxed{T_n = 13.05} \text{ kip/strip}$$

$$T_r = \phi_t\ T_n \qquad \boxed{T_r = 9.79} \text{ kip/strip}$$

Determine the number of soil reinforcing strips based on tensile resistance, N_t

$$N_t = \frac{T_{max2}}{T_r} \qquad \boxed{N_t = 1.38} \text{ strips}$$

E14-2.6.5 Establish Number of Soil Reinforcing Strips at Z

$N_p = 1.48$ Based on pullout resistance, strips

$N_t = 1.38$ Based on tensile resistance, strips

Required number of strip reinforcements for each panel width (round up), N_g

$$N_g = ceil(max(N_t, N_p)) \qquad \boxed{N_g = 2} \text{ strips}$$

Calculate the horizontal spacing of reinforcement, S_h, at Z by dividing the panel width by the required number of strip reinforcements N_g.

$$S_h = \frac{w_p}{N_g} \qquad \boxed{S_h = 2.50} \text{ ft}$$

Note: The typical horizontal reinforcement spacing, S_h, will be provided at 2.5 ft. This will also be the maximum allowed spacing while satisfying the maximum spacing requirement of 2.7 ft. If the wall requires additional reinforcement the vertical spacing will be maintained and adjustments will be made to the horizontal spacing accordingly.

E14-2.7 Summary of Results

E14-2.7.1 Summary of External Stability

Based on the defined project parameters, embedment depth and length of reinforcement the following external stability checks have been satisfied:

External Check	CDR
Sliding	1.00
Eccentricity	2.12
Bearing	1.37

Table E14-2.7-1
Summary of External Stability Computations

E14-2.7.2 Summary of Internal Stability

Computations for the required number of strip reinforcements at each level is presented in **Table E14-2.7-2**.

		Pullout			Rupture						
Layer	Z	σ_H	T_{max1}	P_{rr}	σ_H	T_{max2}	T_r	N_p	N_t	N_g	S_h
1	0.75	0.46	4.55	5.86	0.53	5.34	9.79	0.78	0.54	2	2.50
2	3.25	0.64	8.05	7.08	0.72	9.00	9.79	1.14	0.92	2	2.50
3	5.75	0.84	10.47	7.98	0.91	11.38	9.79	1.31	1.16	2	2.50
4	**8.25**	**1.01**	**12.67**	**8.54**	**1.08**	**13.55**	**9.79**	**1.48**	**1.38**	**2**	**2.50**
5	10.75	1.17	14.65	9.37	1.24	15.49	9.79	1.56	1.58	2	2.50
6	13.25	1.31	16.42	10.13	1.38	17.22	9.79	1.62	1.76	2	2.50
7	15.75	1.44	17.96	10.46	1.50	18.73	9.79	1.72	1.91	2	2.50
8	18.25	1.54	19.29	10.25	1.60	20.01	9.79	1.88	2.04	3	1.67
9	20.75	1.67	20.84	10.22	1.72	21.55	9.79	2.04	2.20	3	1.67

Table E14-2.7-2
Summary of Internal Stability Computation for Strength I Load Combinations

E14-2.7.3 Element Facings and Drainage Design

The design of element facings will not be examined in this example, but shall be considered in the design. This is to be performed by the wall supplier. This includes, but is not limited to, the structural integrity of the concrete face panels, connections, joint widths, differential settlements and the design of bearing pads used to prevent or minimize point loadings or stress concentrations and to accommodate for small vertical deformations of the panels.

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are meet by including a wrapped pipe underdrain behind the retaining wall as shown in Figure E14-2.8-1.

E14-2.8 Final MSE Wall Schematic

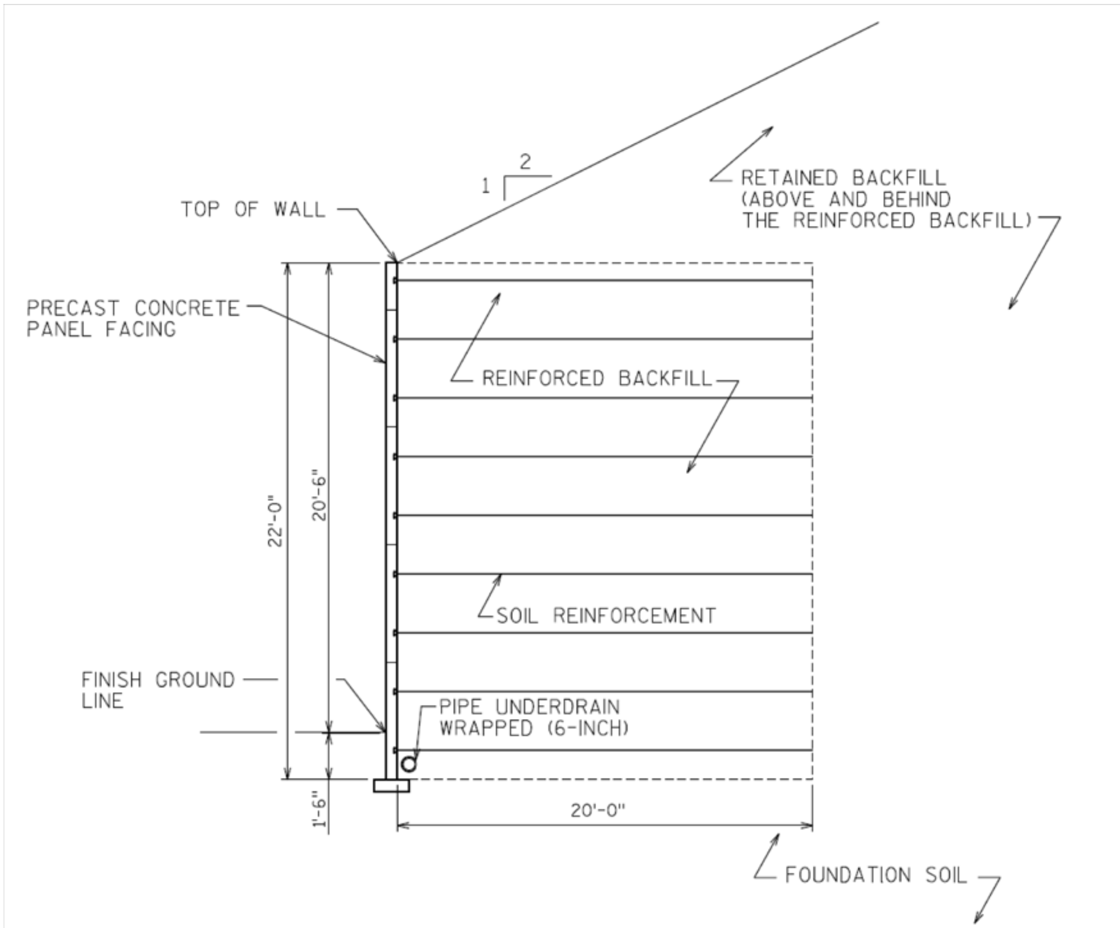

Figure E14-2.8-1
MSE Wall Schematic

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E14-3 Modular Block Facing Geogrid Reinforced MSE Wall, LRFD

General

This example shows design calculations for MSE wall with modular block facings conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for external stability (sliding, eccentricity and bearing) and internal stability (soil reinforcement stress and pullout) will be presented. The overall stability, settlement and connection calculations will not be shown in this example, but are required.

Design steps presented in 14.6.3.3 are used for the wall design.

E14-3.1 Establish Project Requirements

The following MSE wall shall have compacted freely draining soil in the reinforced zone and will be reinforced with geosynthetic (extensible) strips as shown in Figure E14-3.1-1. External stability is the designer's (WisDOT/Consultant) responsibility and internal stability and structural components are the contractors responsibility.

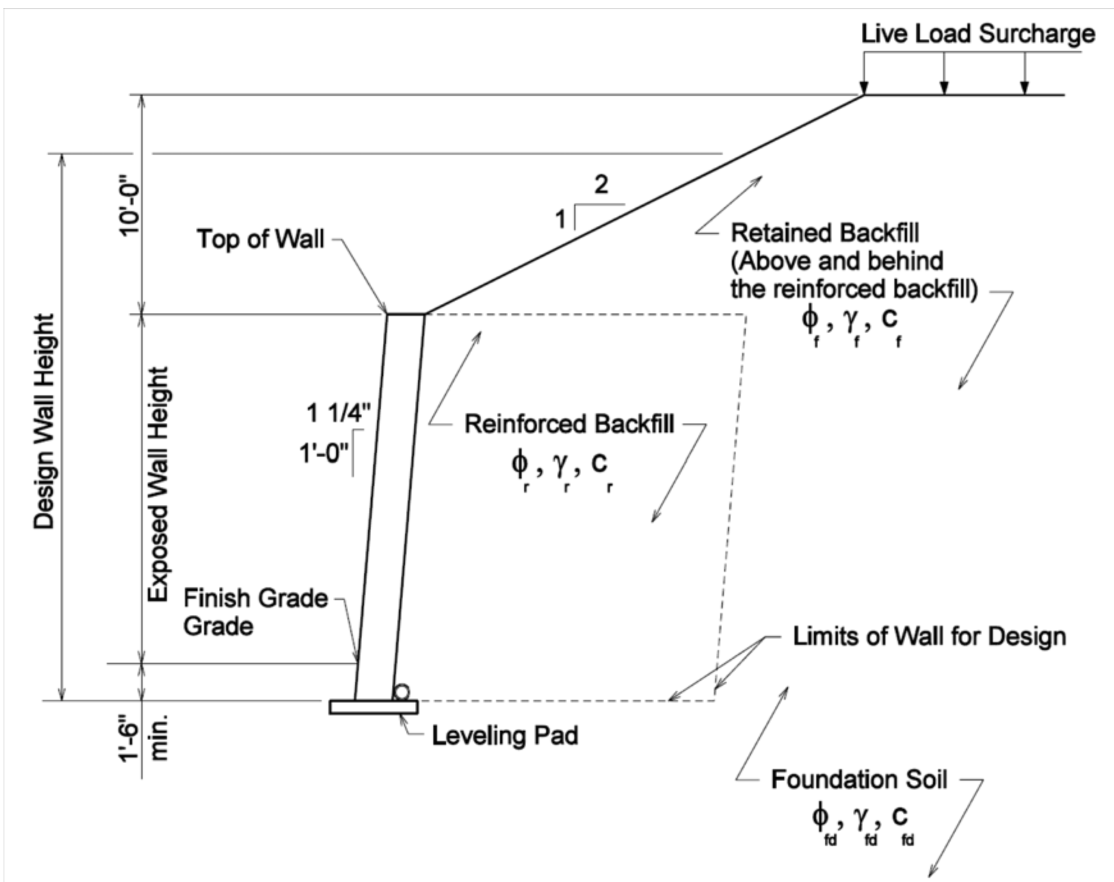

Figure E14-3.1-1
MSE Wall with Broken Backslope & Traffic

Wall Geometry

$H_e = 14.5$ Exposed wall height, ft

$H = H_e + 1.5$ Design wall height, ft (assume 1.5 ft wall embedment)

$\beta = 26.565\ deg$ Inclination of ground slope behind face of wall (2H:1V)

$b_1 = 1.25$ Front wall batter, in/ft (b_1H:12V)

$h_{slope} = 10.0$ Slope height, ft

$Batter = atan\left(\frac{b_1}{12}\right)$ Angle of front face of wall to vertical

$\boxed{Batter = 5.95\ deg}$

Note: Since the wall has less than 10 degrees of batter the wall can be defined as "near vertical" thus θ = 90 degrees and β'=δ'=I for a broken backslope

$\theta = 90\ deg$ Angle of back face of wall to horizontal

$I = atan\left(\frac{h_{slope}}{2\ H}\right)$ Infinite slope angle

$\boxed{I = 17.4\ deg}$

$\beta' = I$ Inclination of ground slope behind face of wall, deg

$\delta' = I$ Friction angle between fill and wall, deg

E14-3.2 Design Parameters

Project Parameters

$Design\_Life = 75$ Wall design life, years (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Reinforced Backfill Soil Design Parameters

$\phi_r = 30\ deg$ Angle of internal friction **LRFD [11.10.5.1]** and (14.4.6)

$\gamma_r = 0.120$ Unit of weight, kcf

$c_r = 0$ Cohesion, psf

Retained Backfill Soil Design Parameters

$\phi_f = 29\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit of weight, kcf

$c_f = 0$ Cohesion, psf

Foundation Soil Design Parameters

$\phi_{fd} = 31deg$ Angle of internal friction

$\gamma_{fd} = 0.125$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, psf

Factored Bearing Resistance of Foundation Soil

$q_R = 6.5$ Factored resistance at the strength limit state, ksf

Note: The factored bearings resistance, q_R, was assumed to be given in the Site Investigation Report. If not provided q_R shall be determined by calculating the nominal bearing resistance, q_n, per **LRFD [Eq 10.6.3.1.2a-1]** and factored with the bearing resistance factor,ϕ_b, for MSE walls (i.e., $q_R=\phi_b q_n$).

Precast Concrete Panel Facing Parameters

$S_v = 1.333$ Vertical spacing of reinforcement, ft

Note: vertical spacing should not exceed 2.7 ft without full scale test data **LRFD [11.10.6.2.1]**

Soil Reinforcement Design Parameters

Geosynthetic - Geogrids Reinforcing type

Note: Product specific information to be defined during internal stability checks

Live Load Surcharge Parameters

$h_{eq} = 2.0$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

$SUR = h_{eq}\,\gamma_f$ Live load soil for surcharge load

$\boxed{SUR = 0.240}$ ksf/ft

Resistance Factors

$\phi_s = 1.00$ Sliding of MSE wall at foundation **LRFD [Table 11.5.7-1]**

$\phi_b = 0.65$ Bearing resistance **LRFD [Table 11.5.7-1]**

$\phi_t = 0.90$ Tensile resistance (geosynthetic reinforcement and connectors) **LRFD [Table 11.5.7-1]**

$\phi_p = 0.90$ Pullout resistance **LRFD [Table 11.5.7-1]**

E14-3.3 Estimate Depth of Embedment and Length of Reinforcement

For this example it is assumed that global stability does not govern the required length of soil reinforcement.

Embedment Depth, d_e

Frost-susceptible material is assumed to be not present or that it has been removed and replaced with nonfrost susceptible material per **LRFD [11.10.2.2]**. There is also no potential for scour. Therefore, the minimum embedment, d_e, shall be the greater of 1.5 ft (14.6.4) or H/20 **LRFD [Table C11.10.2.2-1]**

Note: While AASHTO allows the d_e value of 1.0 ft on level ground, the embedment depth is limited to 1.5 ft by WisDOT policy as stated in Chapter 14.

$$\frac{H}{20} = 0.8 \text{ ft}$$

$$d_e = \max\left(\frac{H}{20}, 1.5\right) \qquad \boxed{d_e = 1.50} \text{ ft}$$

Therefore, the initial design wall height assumption was correct.

$$H_e = 14.5 \text{ ft}$$

$$H = H_e + 1.5 \qquad \boxed{H = 16.00} \text{ ft}$$

Length of Reinforcement, L

In accordance with **LRFD [11.10.2.1]** the minimum required length of soil reinforcement shall be the greater of 8 feet or 0.7H. Due to the sloping backfill and traffic surcharge a longer reinforcement length of 0.9H will be used in this example. The length of reinforcement will be uniform throughout the entire wall height.

$$0.9\,H = 14.4 \text{ ft}$$

$$L_{user} = 14.5 \text{ ft}$$

$$L = \max(8.0, 0.9\,H, L_{user}) \qquad \boxed{L = 14.50} \text{ ft}$$

Height of retained fill at the back of the reinforced soil, h

$$h = H + L\tan(\beta) \qquad \boxed{h = 23.25} \text{ ft}$$

E14-3.4 Permanent and Transient Loads

In this example, load types EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used as shown in Figure E14-3.4-1. No transient loads are present in this example. Due to the relatively thin wall thickness the weight and width of the concrete facing will be ignored. Passive soil resistance will also be ignored.

E14-3.4.1 Compute Active Earth Pressure

Compute the coefficient of active earth pressure (k_a) using Coulomb Theory **LRFD [Eq 3.11.5.3-1]** with the wall backfill material interface friction angle, δ, set equal to β (i.e. δ=β) **LRFD [11.10.5.2].** The retained backfill soil will be used (i.e., k_a=k_{af})

$$\phi_f = 29 \text{ deg}$$

$$\beta' = 17.4 \text{ deg}$$

$$\theta = 90 \text{ deg}$$

$$\delta' = 17.4 \text{ deg}$$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta')\ \sin(\phi_f - \beta')}{\sin(\theta - \delta')\ \sin(\theta + \beta')}}\right)^2 \qquad \boxed{\Gamma = 1.961}$$

$$k_{af} = \frac{\sin(\theta + \phi_f)^2}{\Gamma\ \sin(\theta)^2\ \sin(\theta - \delta')} \qquad \boxed{k_{af} = 0.409}$$

E14-3.4.2 Compute Unfactored Loads

The forces and moments are computed using Figure E14-3.4-1 by their appropriate LRFD load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**

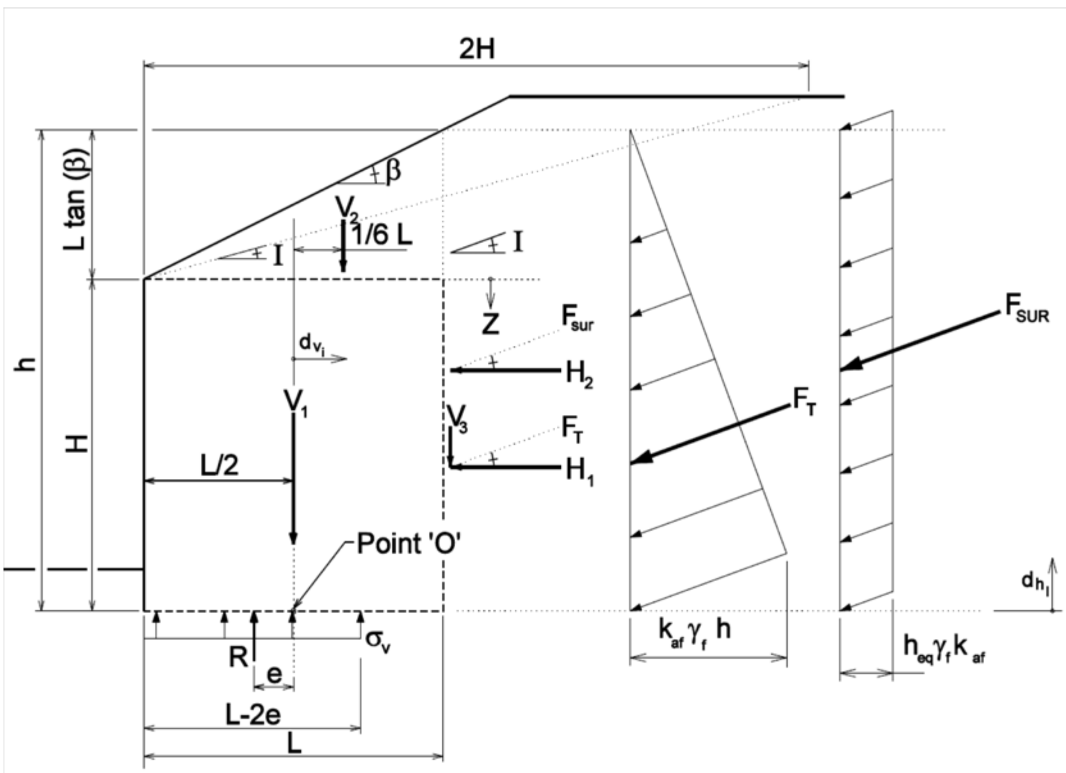

Figure E14-3.4-1
MSE Wall - External Stability

Active Earth Force Resultant, (kip/ft), F_T

$F_T = \frac{1}{2} \gamma_f h^2 k_{af}$ Active earth force resultant (EH) $\boxed{F_T = 13.3}$

Live Load Surcharge, (kip/ft), F_{SUR}

$F_{SUR} = h_{eq} \gamma_f h k_{af}$ Live load surcharge resultant (LS) $\boxed{F_{SUR} = 2.3}$

Vertical Loads, (kip/ft), V_i

$V_1 = \gamma_r H L$ Soil backfill - reinforced soil (EV) $\boxed{V_1 = 27.8}$

$V_2 = \frac{1}{2} \gamma_f L (L \tan(\beta))$ Soil backfill - backslope (EV) $\boxed{V_2 = 6.3}$

$V_3 = F_T \sin(I)$ Active earth force resultant (vertical component - EH) $\boxed{V_3 = 4}$

Moments produced from vertical loads about the center of reinforced soil, (kip-ft/ft) MV_i

Moment Arm		Moment	
$d_{v1} = 0$	$\boxed{d_{v1} = 0.0}$	$MV_1 = V_1 d_{v1}$	$\boxed{MV_1 = 0.0}$
$d_{v2} = \frac{1}{6} L$	$\boxed{d_{v2} = 2.4}$	$MV_2 = V_2 d_{v2}$	$\boxed{MV_2 = 15.2}$
$d_{v3} = \frac{L}{2}$	$\boxed{d_{v3} = 7.3}$	$MV_3 = V_3 d_{v3}$	$\boxed{MV_3 = 28.7}$

Horizontal Loads, (kip/ft), H_i

$H_1 = F_T \cos(I)$ Active earth force resultant (horizontal component - EH) $\boxed{H_1 = 12.7}$

$H_2 = F_{SUR} \cos(I)$ Live load surcharge resultant (LS) $\boxed{H_2 = 2.2}$

Moments produced from horizontal loads about the center of reinforced soil, (kip-ft/ft), MH

Moment Arm Moment

$d_{h1} = \frac{h}{3}$ $\boxed{d_{h1} = 7.7}$ $MH_1 = H_1\, d_{h1}$ $\boxed{MH_1 = 98.0}$

$d_{h2} = \frac{h}{2}$ $\boxed{d_{h2} = 11.6}$ $MH_2 = H_2\, d_{h2}$ $\boxed{MH_2 = 25.3}$

Summary of Unfactored Forces & Moments:

Load			**Moment Arm**		**Moment**		**LRFD Load Type**
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Soil backfill	27.80	d_{v1}	0.0	MV_1	0.0	EV
V_2	Soil backfill	6.30	d_{v2}	2.4	MV_2	15.2	EV
V_3	Active earth pressure	4.00	d_{v3}	7.3	MV_3	28.7	EH

Table E14-3.4-1
Unfactored Vertical Forces & Moments

Load			**Moment Arm**		**Moment**		**LRFD Load Type**
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Active earth pressure	12.70	d_{h1}	7.7	MH_1	98.0	EH
H_2	Live Load Surcharge	2.20	d_{h2}	11.6	MH_2	25.3	LS

Table E14-3.4-2
Unfactored Horizontal Forces & Moments

E14-3.4.3 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all load modifiers to one ($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be used in this example:

Load Combination Limit State	EV	LS	EH
Strength Ia (minimum)	$\gamma_{EVmin} = 1.00$	$\gamma_{LSmin} = 1.75$	$\gamma_{EHmin} = 0.90$
Strength Ib (maximum)	$\gamma_{EVmax} = 1.35$	$\gamma_{LSmax} = 1.75$	$\gamma_{EHmax} = 1.50$
Service I (max/min)	$\gamma_{EV} = 1.00$	$\gamma_{LS} = 1.00$	$\gamma_{EH} = 1.00$

Load Combination	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.35	1.75	1.75	1.50	Bearing, T_{max}
Service I	1.00	1.00	1.00	1.00	Pullout (σ_v)

Table E14-3.4-3
Load Combinations

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)}$ = 0.9, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_3\gamma_{EH(max)}$ and $H_1\gamma_{EH(max)}$ or $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(min)}$, not $V_3\gamma_{EH(min)}$ and $H_1\gamma_{EH(max)}$.
- T_{max1} (Pullout) is calculated without live load and T_{max2} (Rupture) is calculated with live load.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-3.4.4 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$$V_{EV} = V_1 + V_2$$ $\boxed{V_{EV} = 34.1}$

$$V_{EH} = V_3$$ $\boxed{V_{EH} = 4.0}$

$$H_{EH} = H_1$$ $\boxed{H_{EH} = 12.7}$

$$H_{LS} = H_2$$ $\boxed{H_{LS} = 2.2}$

Unfactored moments by load type (kip-ft/ft)

$$M_{EV} = MV_1 + MV_2$$ $\boxed{M_{EV} = 15.2}$

$$M_{EH1} = MV_3$$ $\boxed{M_{EH1} = 28.7}$

$$M_{EH2} = MH_1$$ $\boxed{M_{EH2} = 98.0}$

$$M_{LS2} = MH_2$$ $\boxed{M_{LS2} = 25.3}$

Factored vertical loads by limit state (kip/ft)

$$V\_Ia = n\left(1.00V_{EV} + 1.50\ V_{EH}\right)$$ $\boxed{V\_Ia = 40.1}$

$$V\_Ib = n\left(1.35V_{EV} + 1.50\ V_{EH}\right)$$ $\boxed{V\_Ib = 52.0}$

$$V\_Ser = n\left(1.00V_{EV} + 1.00\ V_{EH}\right)$$ $\boxed{V\_Ser = 38.1}$

Factored horizontal loads by limit state (kip/ft)

$$H\_Ia = n\left(1.75H_{LS} + 1.50H_{EH}\right)$$ $\boxed{H\_Ia = 22.8}$

$$H\_Ib = n\left(1.75H_{LS} + 1.50H_{EH}\right)$$ $\boxed{H\_Ib = 22.8}$

$$H\_Ser = n\left(1.00H_{LS} + 1.00H_{EH}\right)$$ $\boxed{H\_Ser = 14.8}$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$$MV\_Ia = n\left(1.00M_{EV} + 1.50\ M_{EH1}\right)$$ $\boxed{MV\_Ia = 58.2}$

$$MV\_Ib = n\left(1.35M_{EV} + 1.50\ M_{EH1}\right)$$ $\boxed{MV\_Ib = 63.6}$

$$MV\_Ser = n\left(1.00M_{EV} + 1.00\ M_{EH1}\right)$$ $\boxed{MV\_Ser = 43.9}$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$$MH\_Ia = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right)$$ $\boxed{MH\_Ia = 191.3}$

$$MH\_Ib = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right)$$ $\boxed{MH\_Ib = 191.3}$

$$MH\_Ser = n\left(1.00M_{LS2} + 1.00\ M_{EH2}\right)$$ $\boxed{MH\_Ser = 123.3}$

Summary of Factored Forces & Moments:

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	40.1	58.2	22.8	191.3
Strength Ib	52.0	63.6	22.8	191.3
Service I	38.1	43.9	14.8	123.3

Table E14-3.4-4
Summary of Factored Loads & Moments

E14-3.5 Evaluate External Stability of MSE Wall

Three potential external failure mechanisms will be considered in this example (sliding at the base, limiting eccentricity and bearing resistance). Overall (global) stability requirements are not included here. Design calculations will be carried out for the governing limit states only.

E14-3.5.1 Sliding Resistance at Base of MSE Wall

The following calculations are based on **Strength Ia**:

Factored Sliding Force

$R_u = H\_Ia$ $\boxed{R_u = 22.8}$ kip/ft

Sliding Resistance

To compute the coefficient of sliding friction for continuous reinforcement use the lesser friction angle of the reinforced back fill, ϕ_r , or the foundation soil, ϕ_{fd}, **LRFD [11.10.5.3]**.

$\phi_\mu = \min(\phi_r, \phi_{fd})$ $\boxed{\phi_\mu = 30 \text{ deg}}$

Note: Since continuous reinforcement is used, a slip plane may occur at the reinforcement layer. The sliding friction angle for this case shall use the lesser of (when applicable) ϕ_r, ϕ_{fd}, and ρ. Where ρ is the soil-reinforcement interface friction angle. Without specific data ρ may equal 2/3 ϕ_f with ϕ_f a maximum of 30 degrees. This check is not made in this example, but is required.

$\mu = \tan(\phi_\mu)$ $\boxed{\mu = 0.577}$

$V\_Ia = 40.1$ Factored vertical load, kip/ft

$V_{Nm} = \mu\ V\_Ia$ $\boxed{V_{Nm} = 23.1}$ kip/ft

$\phi_s = 1.00$

$R_R = \phi_s V_{Nm}$ $\boxed{R_R = 23.1}$ kip/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} = \frac{R_R}{R_u}$$ $\boxed{CDR_{Sliding} = 1.02}$

Is the CDR ≥ 1.0 ? $\boxed{check = \text{"OK"}}$

E14-3.5.2 Limiting Eccentricity at Base of MSE Wall

The location of the resultant of the reaction forces is limited to the middle two-thirds of the base width for a soil foundation (i.e., e_{max} = L/3) **LRFD [11.6.3.3]**. The following calculations are based on **Strength Ia**.

Maximum eccentricity

$$e_{max} = \frac{L}{3}$$ $\boxed{e_{max} = 4.83}$ ft

Compute wall eccentricity (distance from Point 'O' Figure E14-3.4-1)

$\Sigma M_R = MV\_Ia$ Summation of resisting moments for Strength Ia

$\Sigma M_O = MH\_Ia$ Summation of overturning moments for Strength Ia

$\Sigma V = V\_Ia$ Summation of vertical loads for Strength Ia

$\boxed{\Sigma M_R = 58.2}$ kip-ft/ft

$\boxed{\Sigma M_O = 191.3}$ kip-ft/ft

$\boxed{\Sigma V = 40.1}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$ $\boxed{e = 3.32}$ ft

Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} = \frac{e_{max}}{e}$$ $\boxed{CDR_{Eccentricity} = 1.46}$

Is the CDR ≥ 1.0 ? $\boxed{check = \text{"OK"}}$

E14-3.5.3 Bearing Resistance at base of MSE Wall

The following calculations are based on **Strength Ib**:

Compute wall eccentricity (distance from Point 'O' Figure E14-3.4-1)

$\Sigma M_R = MV\_Ib$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ Summation of vertical loads for Strength Ib

$\boxed{\Sigma M_R = 63.6}$ kip-ft/ft

$\boxed{\Sigma M_O = 191.3}$ kip-ft/ft

$\boxed{\Sigma V = 52.0}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$

$\boxed{e = 2.46}$ ft

Compute the ultimate bearing stress

σ_V = Ultimate bearing stress
L = Bearing length
e = Eccentricity (resultant produced by extreme bearing resistance loading)

Note: For the bearing resistance calculations the effective bearing width, B' = L-2e, is used instead of the actual width. Also, when the eccentricity, e, is negative: B'=L. The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**.

$$\sigma_V = \frac{\Sigma V}{L - 2\,e}$$

$\boxed{\sigma_V = 5.43}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 6.50}$ ksf/ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Bearing} = \frac{q_R}{\sigma_V}$$

$\boxed{CDR_{Bearing} = 1.20}$

Is the $CDR \geq 1.0$?

$\boxed{check = \text{"OK"}}$

E14-3.6 Evaluate Internal Stability of MSE Wall

Note: MSE walls are a proprietary wall system and the internal stability computations shall be performed by the wall supplier.

Internal stability shall be checked for 1) pullout and 2) rupture in accordance with **LRFD [11.10.6]**. The factored tensile load, T_{max}, is calculated twice for internal stability checks for vertical stress (σ_V) calculations. For pullout T_{max1} is determined by excluding live load surcharge. For rupture T_{max2} is determined by including live load surcharge. In this example, the maximum reinforcement loads are calculated using the Simplified Method.

The location of the potential failure surface for a MSE wall with metallic strip or grid reinforcements (inextensible) is shown in Figure E14-2.6-1.

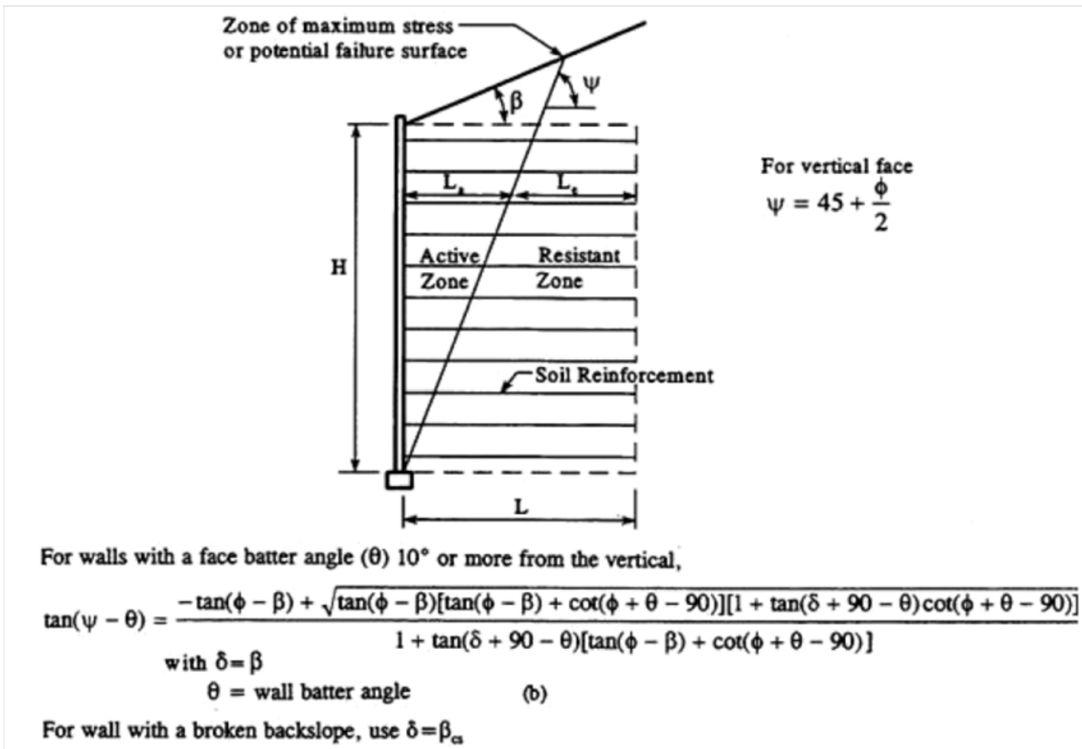

Figure E14-2.6-1
MSE Wall - Internal Stability (Extensible Reinforcement)
FHWA [Figure 4-9]

E14-3.6.1 Establish the Vertical Layout of Soil Reinforcement

Soil reinforcement layouts are shown in Table E14-3.6-1. They were determined by a standard block wall unit thickness of 8-in and a maximum vertical reinforcement spacing of 2.7-ft. The top and bottom level vertical spacing was adjusted to fit the height of the wall. Computations for determining the maximum tension, T_{max}, are taken at each level in the vertical layout.

$Layer = 3$ Layer of reinforcement (from top)

$Z = 3.333$ ft Depth below top of wall, ft

$S_v = 1.33$ ft Vertical spacing of reinforcement, ft

Calculate the upper and lower tributary depths based on the reinforcement vertical spacing

$$Z_{neg} = Z - \frac{S_v}{2}$$ $\boxed{Z_{neg} = 2.67}$ ft

$$Z_{pos} = Z + \frac{S_v}{2}$$ $\boxed{Z_{pos} = 4.00}$ ft

Layer	Z (ft)	Zneg (ft)	Zpos (ft)
1	0.67	0.00	1.33
2	2.00	1.33	2.67
3	3.33	2.67	4.00
4	4.67	4.00	5.33
5	6.00	5.33	6.67
6	7.33	6.67	8.00
7	8.67	8.00	9.33
8	10.00	9.33	10.67
9	11.33	10.67	12.00
10	12.67	12.00	13.33
11	14.00	13.33	14.67
12	15.33	14.67	16.00

Table E14-3.6-1
Vertical Layout of Soil Reinforcement

E14-3.6.2 Compute Horizontal Stress and Maximum Tension, T_{max}

Factored horizontal stress

$$\sigma_H = \gamma_P (\sigma_v k_r + \Delta\sigma_H)$$ **LRFD [Eq 11.10.6.2.1-1]**

γ_p = Load factor for vertical earth pressure (γ_{EVmax})

k_r = Horizontal pressure coefficient

σ_v = Pressure due to gravity and surcharge for pullout, T_{max1} ($\gamma_r Z_{trib} + \sigma_2$)

σ_v = Pressure due to gravity and surcharge for pullout resistance ($\gamma_r Z_{p\text{-}PO}$)

σ_v = Pressure due to gravity and surcharge for rupture, T_{max2} ($\gamma_r Z_{trib} + \sigma_2 + q$)

$\Delta\sigma_H$ = Horizontal pressure due to concentrated horizontal surcharge load

Z = Reinforcement depth for max stress Figure E14-2.6-2

Z_p = Depth of soil at reinforcement layer potential failure plane

$Z_{p\text{-}ave}$ = Average depth of soil at reinforcement layer in the effective zone

σ_2 = Equivalent uniform stress from backslope ($0.5(0.7)L\tan\beta)\gamma_f$

q = Surcharge load ($q = SUR$), ksf

To compute the lateral earth pressure coefficient, k_r, a k_a multiplier is used to determine k_r for each of the respective vertical tributary spacing depths (Z_{pos}, Z_{neg}). The k_a multiplier is determined using Figure E14-2.6-2. To calculate k_a it is assumed that $\delta=\beta$ and $\beta = 0$; thus, $k_a=\tan^2(45-\phi_f/2)$ **LRFD [Eq C11.10.6.2.1-1]**

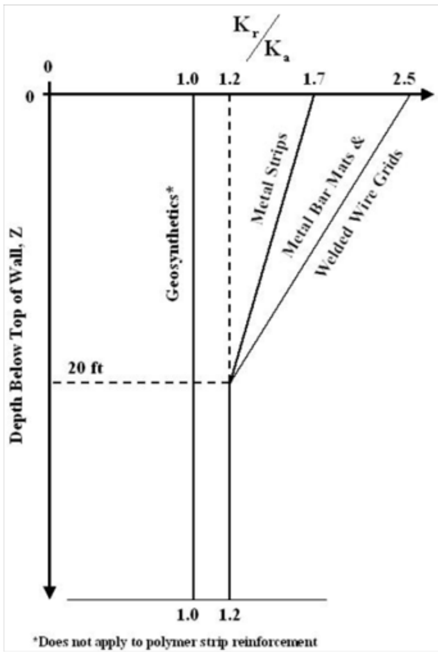

Figure E14-3.6-2
k_r/k_a Variation with MSE Wall Depth
FHWA [Figure 4-10]

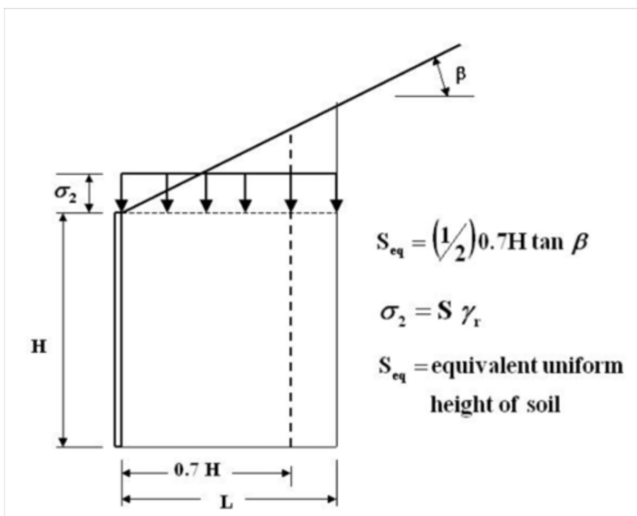

Figure E14-3.6-3
Calculation of Vertical Stress
FHWA [Figure 4-11]

Calculate the coefficient of active earth pressure, k_a

$$\phi_r = 30 \text{ deg}$$

$$k_a = \tan\left(45 \text{ deg} - \frac{\phi_r}{2}\right)^2 \qquad \boxed{k_a = 0.333}$$

The internal lateral earth pressure coefficient, k_r, for geogrids remains constant throughout the reinforced soil zone. k_r will be equal to $k_a(k_r/k_a) = k_a$ at any depth below the top of wall as shown in figure E14-3.6-2 **LRFD [Figure 11.10.6.2.2-3]**.

$$k_r = k_a \qquad \boxed{k_r = 0.333}$$

Compute effective (resisting) length, L_e

$Z = 3.33$ ft

$H = 16.00$ ft

$L = 14.5$ ft

$$\psi = 45 \text{ deg} + \frac{\phi_r}{2} \qquad \boxed{\psi = 60.0 \text{ deg}}$$

$$L_a = \frac{H - Z}{\tan(\psi)} \qquad \boxed{L_a = 7.31}$$

$$L_e = \max(L - L_a, 3) \qquad \boxed{L_e = 7.19}$$

Note: L_e shall be greater than or equal to 3 ft **LRFD [11.10.6.3.2]**

E14-3.6.3 Establish Nominal and Factored Pullout Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H, at Z by averaging the upper and lower tributary values (Z_{neg} and Z_{pos}). Since there is no horizontal stresses from concentrated dead loads values $\Delta\sigma_H$ is set to zero.

$$\sigma_H = \gamma_{EVmax}\,(\gamma_r Z_{trib} + \sigma_2)k_r$$

Surcharge loads

$$\sigma_2 = \frac{1}{2}\,0.7\,H\,\tan(\beta)\,\gamma_f$$ $\boxed{\sigma_2 = 0.336}$ ksf/ft

Horizontal stress at Z_{neg} and Z_{pos}

$$\sigma_{H\_neg} = \gamma_{EVmax}\left(\gamma_r Z_{neg} + \sigma_2\right) k_r$$ $\boxed{\sigma_{H\_neg} = 0.295}$ ksf/ft

$$\sigma_{H\_pos} = \gamma_{EVmax}\left(\gamma_r Z_{pos} + \sigma_2\right) k_r$$ $\boxed{\sigma_{H\_pos} = 0.367}$ ksf/ft

Horizontal stress at Z

$$\sigma_H = 0.5\left(\sigma_{H\_pos} + \sigma_{H\_neg}\right)$$ $\boxed{\sigma_H = 0.331}$ ksf/ft

Compute the maximum tension, $T_{max\,1}$, at Z

$S_v = 1.33$ ft

$$T_{max1} = \sigma_H\,S_v\,1000.$$ $\boxed{T_{max1} = 441}$ plf

Compute effective vertical stress for pullout resistance, σ_v

$$Z_{p\_PO} = Z + 0.5\,\tan(\beta)\,\left(L_a + L\right)$$ $\boxed{Z_{p\_PO} = 8.8}$ ft

$\gamma_{EV} = 1.00$ Unfactored vertical stress for pullout resistance **LRFD [11.10.6.3.2]**

$$\sigma_v = \gamma_{EV}\,\gamma_r\,Z_{p\_PO}\,1000$$ $\boxed{\sigma_v = 1054}$ psf

Compute pullout resistance factor, F*

Pullout resistance factor, F*, for extensible geosynthetic reinforcement remains constant throughout the reinforced soil for determining the internal lateral earth pressure. Since no product-specific pullout test data is provided at the time of design F* and the scale effect correction factor,α, default values will be used per **LRFD [Figure 11.10.6.3.2-1 and Table 11.10.6.3.2-1]**.

Use default values for F' and α since product-specific pullout test data has not been provided.

$$F' = 0.67\,\tan\left(\phi_r\right)$$ Pullout Friction Factor (Geogrids F*= 0.67tanϕ'_r Default value) **LRFD [Figure 11.10.6.3.2-1]**

$\boxed{F' = 0.387}$

Compute nominal pullout resistance, P_r

$\alpha = 0.8$ — Scale effect correction factor (geogrids α = 0.8 default value) **LRFD [Table 11.10.6.3.2-1]**

$C = 2$ — Overall reinforcement surface area geometry factor (geogrids C = 2.0) **LRFD [11.10.6.3.2]**

$R_c = 1$ — Reinforcement coverage ratio (continuous reinforcement R_c = 1.0) **LRFD [11.10.6.4]**

$P_r = F' \alpha \sigma_v C R_c L_e$ $\boxed{P_r = 4690}$ plf

Compute factored pullout resistance, P_{rr}

$\phi_p = 0.9$

$P_{rr} = \phi_p P_r$ $\boxed{P_{rr} = 4221}$ plf

E14-3.6.4 Establish Nominal and Factored Long-term Tensile Resistance of Soil Reinforcement

Compute the factored horizontal stress, σ_H

$$\sigma_H = \gamma_{EVmax} (\gamma_r Z + \sigma_2 + q) k_r$$

Surcharge loads

$\boxed{\sigma_2 = 0.34}$ ksf/ft

$\boxed{q = 0.24}$ ksf/ft

Horizontal stress at Z_{neg} and Z_{pos}

$\sigma_{H\_neg} = \gamma_{EVmax} \left(\gamma_r Z_{neg} + \sigma_2 + q\right) k_r$ $\boxed{\sigma_{H\_neg} = 0.40}$ ksf/ft

$\sigma_{H\_pos} = \gamma_{EVmax} \left(\gamma_r Z_{pos} + \sigma_2 + q\right) k_r$ $\boxed{\sigma_{H\_pos} = 0.48}$ ksf/ft

Horizontal stress at Z

$\sigma_H = 0.5\left(\sigma_{H\_pos} + \sigma_{H\_neg}\right)$ $\boxed{\sigma_H = 0.44}$ ksf/ft

Compute the maximum tension, T_{max2}, at Z

$S_v = 1.33$ ft

$T_{max2} = \sigma_H S_v 1000$ $\boxed{T_{max2} = 585}$ plf

$$T_r=\phi T_{al}=\phi T_{ult}/RF$$

T_r = Factored soil reinforcement tensile resistance
ϕ = Resistance factor
T_{al} = Nominal geosynthetic reinforcement strength
T_{ult} = Ultimate tensile strength
RF_{CR} = Creep reduction factor
RF_D = Durability reduction factor
RF_{ID} = Installation damage reduction factor
RF = Reduction factor (RF_{CR} x RF_D x RF_{ID})

The following calculation for determining the nominal long-term reinforcement tensile strength uses values similar to proprietary product specific data. In any application RF_{ID} nor RF_D shall not be less than 1.1. A single default reduction factor, RF, of 7 may be used for permanent applications if meeting the requirements listed in **LRFD [11.10.6.4.2b and Table 11.10.6.4.2b-1,Table 11.10.6.4.2b-1]**

	Geogrid Type		
	#1	#2	#3
T_{ult} (plf)	2500	5000	7500
RF_{CR}	2.00	2.00	2.00
RF_D	1.15	1.15	1.15
RF_{ID}	1.35	1.35	1.35

Table E14-3.6-2
Geogrid Resistance Properties

$\text{Grade} = 1$

$T_{ult} = 2500$ plf

$RF_{CR} = 2.00$

$RF_D = 1.15$

$RF_{ID} = 1.35$

$RF = RF_{CR}\, RF_D\, RF_{ID}$ $\qquad \boxed{RF = 3.11}$

$T_{al} = \dfrac{T_{ult}}{RF}$ $\qquad \boxed{T_{al} = 805}$ plf

$T_r = \phi_t\, T_{al}$ $\qquad \boxed{T_r = 725}$ plf

E14-3.6.5 Establish Grade of Soil Reinforcing Elements at Each Level

Based on Pullout Resistance

$$CDR_{pullout} = \frac{P_{rr}}{T_{max1}}$$ $\boxed{CDR_{pullout} = 9.56}$

Is the CDR $\geq$ 1.0 ? $\boxed{check = \text{"OK"}}$

Based on Tensile Resistance

$$CDR_{tensile} = \frac{T_r}{T_{max2}}$$ $\boxed{CDR_{tensile} = 1.24}$

Is the CDR $\geq$ 1.0 ? $\boxed{check = \text{"OK"}}$

Note: If the wall requires additional reinforcement the vertical spacing will be maintained and adjustments will be made to the grade (strength) for each layer accordingly.

E14-3.7 Summary of Results

E14-3.7.1 Summary of External Stability

Based on the defined project parameters, embedment depth and length of reinforcement the following external stability checks have been satisfied:

External Check	CDR
Sliding	1.02
Eccentricity	1.46
Bearing	1.20

Table E14-3.7-1
Summary of External Stability Computations

E14-3.7.2 Summary of Internal Stability

Computations for the grades of geogrid reinforcements at each level is presented in Table E14-3.7-2.

Level	Z	Pullout			Rupture					
		σ_H	T_{max1}	P_{rr}	Grade	σ_H	T_{max2}	T_r	CDR_p	CDR_t
1	0.67	187	250	2455	#1	295	394	725	9.84	1.84
2	2.00	259	346	3280	#1	367	490	725	9.49	1.48
3	3.33	331	442	4221	#1	439	586	725	9.56	1.24
4	4.67	403	538	5280	#1	511	682	725	9.82	1.06
5	6.00	475	634	6456	#2	583	778	1449	10.19	1.86
6	7.33	547	730	7750	#2	655	874	1449	10.62	1.66
7	8.67	619	826	9161	#2	727	970	1449	11.10	1.49
8	10.00	691	922	10690	#2	799	1066	1449	11.60	1.36
9	11.33	763	1018	12336	#2	871	1162	1449	12.12	1.25
10	12.67	835	1114	14099	#2	943	1258	1449	12.66	1.15
11	14.00	907	1210	15980	#2	1015	1354	1449	13.21	1.07
12	15.33	979	1306	17978	#3	1087	1450	2174	13.77	1.50

Table E14-3.7.2
Summary of Internal Stability Computations for Strength I Load Combinations

E14-3.8 Final MSE Wall Schematic

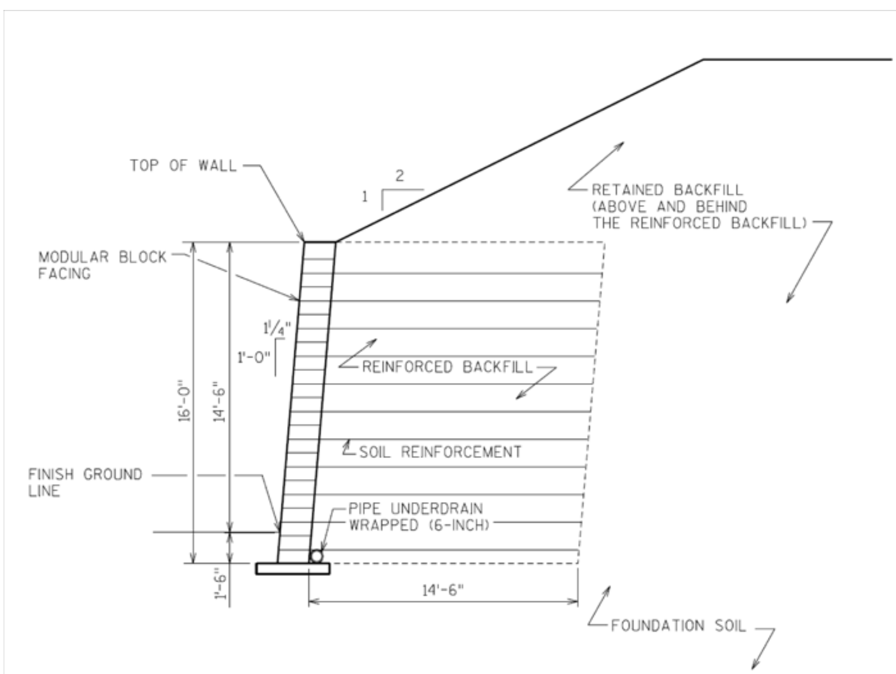

Figure E14-3.8-1
MSE Wall Schematic

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E14-4 Cast-In-Place Concrete Cantilever Wall on Piles, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on piles conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

Sample design calculations for pile capacities and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-4.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-4.1-1 will be designed appropriately to accommodate a horizontal backslope. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

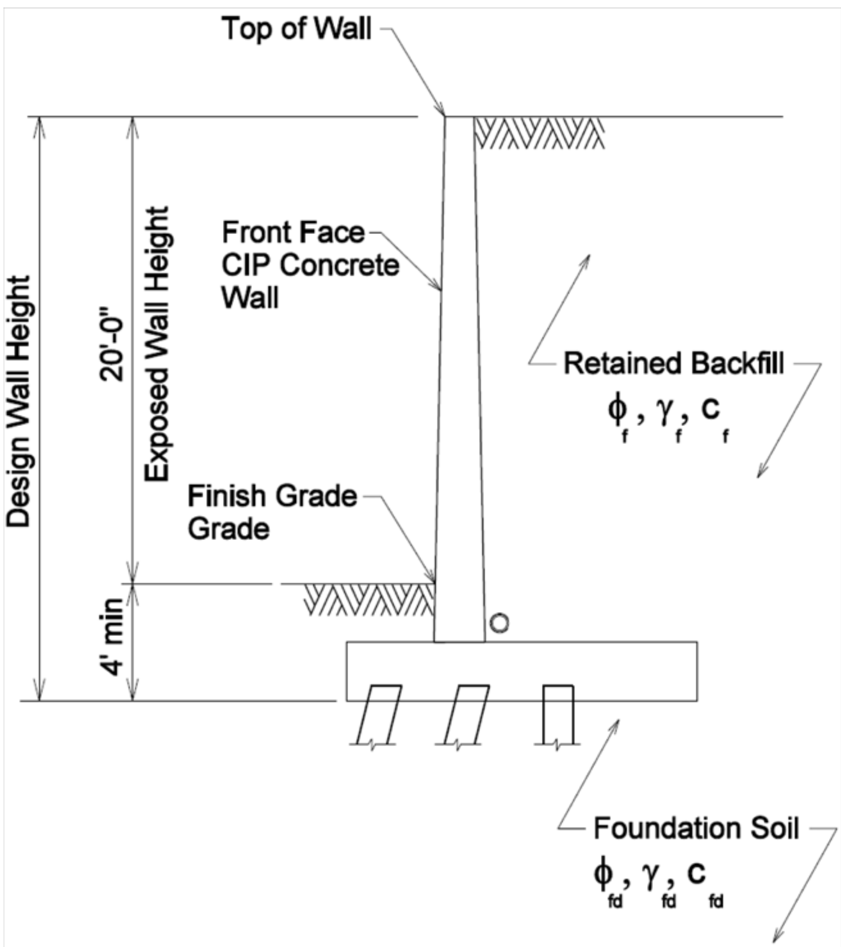

Figure E14-4.1-1
CIP Concrete Wall on Piles

E14-4.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 32\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit weight, kcf

$c_f = 0$ Cohesion, ksf

$\delta = 17\ deg$ Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

ϕ_f= 32 degrees is used for this example, however ϕ_f=30 degrees is the maximum that should be used without testing.

Foundation Soil Design Parameters

$\phi_{fd} = 29\ deg$ Angle of internal friction

$\gamma_{fd} = 0.110$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, ksf

Reinforced Concrete Parameters

$f'_c = 3.5$ Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $w_c = \gamma_c$ Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ Modulus of elasticity of concrete, ksi **LRFD [C5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 100.00$ Distance from wall backface to edge of traffic, ft

$\frac{H}{2} = 12.00$ Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet).

Shall live load surcharge be included? check = "NO"

$h_{eq} = 0.833$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

WisDOT Policy: Wall with live load from traffic use 2.0 feet (240 psf) and walls without traffic use 0.833 feet (100 psf)

E14-4.3 Define Wall Geometry

Wall Geometry

$H_e = 20.00$ Exposed wall height, ft

$D_f = 4.00$ Footing cover, ft (WisDOT policy 4'-0" minimum)

$H = H_e + D_f$ Design wall height, ft

$T_t = 1.00$ Stem thickness at top of wall, ft

$b_1 = 0.25$ Front wall batter, in/ft (b_1H:12V)

$b_2 = 0.50$ Back wall batter, in/ft (b_2H:12V)

$\beta = 0.00$ deg Inclination of ground slope behind face of wall, deg (horizontal)

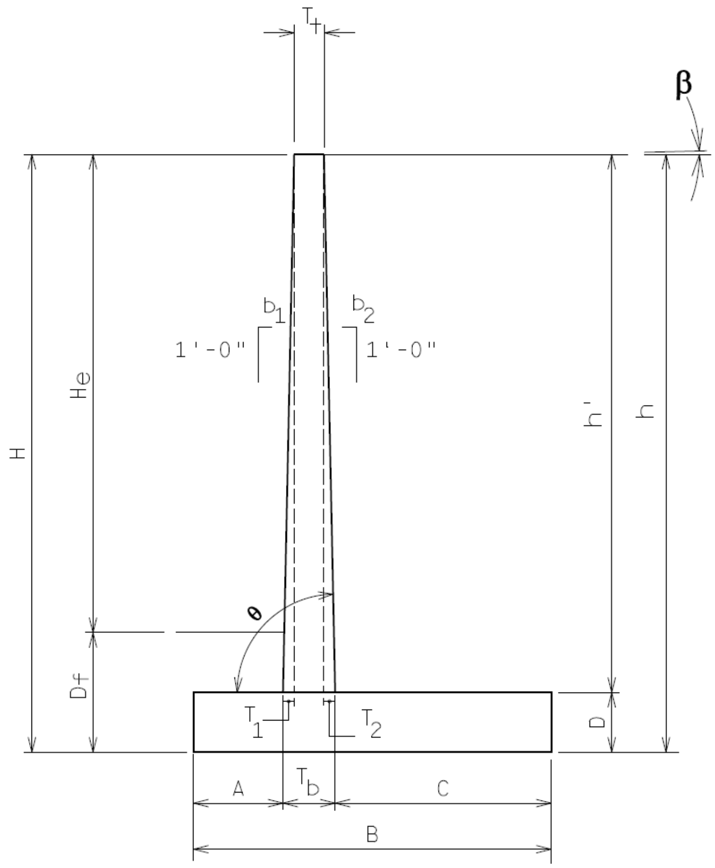

Figure E14-4.3-1
CIP Concrete Wall Geometry

Preliminary Wall Dimensioning

Selecting the most optimal wall configuration is an iterative process and depends on site conditions, cost considerations, wall geometry and aesthetics. For this example, the iterative process has been completed and the final wall dimensions are used for design checks.

$H = 24.0$	Design wall height, ft
$B = 12.00$	Footing base width, ft (2/5H to 3/5H)
$A = 4.75$	Toe projection, ft (H/8 to H/5)
$D = 2.50$	Footing thickness, ft (H/8 to H/5)

WisDOT policy: H≤10'-0" D_{min}=1'-6"
H>10'-0" D_{min}=2'-0"
On Piles D_{min}=2'-0"

Other Wall Dimensioning

$h' = H - D$	Stem height, ft	$h' = 21.5$
$T_1 = b_1 \frac{h'}{12}$	Stem front batter width, ft	$T_1 = 0.448$
$T_2 = b_2 \frac{h'}{12}$	Stem back batter width, ft	$T_2 = 0.896$
$T_b = T_1 + T_t + T_2$	Stem thickness at bottom of wall, ft	$T_b = 2.34$
$C = B - A - T_b$	Heel projection, ft	$C = 4.91$
$\theta = \operatorname{atan}\left(\frac{12}{b_2}\right)$	Angle of back face of wall to horizontal	$\theta = 87.6\ deg$
$b = 12$	Concrete strip width for design, in	
$h = H + (T_2 + C)\tan(\beta)$	Retained soil height, ft	$h = 24.0$

Pile Dimensioning

$y_{p1} = 1.25$	Distance from Point 'O' to centerline pile row 1, ft
$PS1 = 2.75$	Distance from centerline pile row 1 to centerline pile row 2, ft
$PS2 = 3.00$	Distance from centerline pile row 2 to centerline pile row 3, ft
$P_1 = 8.00$	Spacing between piles in row 1, ft
$P_2 = 8.00$	Spacing between piles in row 2, ft
$P_3 = 8.00$	Spacing between piles in row 3, ft

Pile Parameters (From Geotechnical Site Investigation Report, assuming HP12x53)

Pile_Axial = 220	Pile axial capacity (factored), kips
pile_batter = 4	Pile batter (pile_batterV:1H)
$H_{r1} = 11$	Pile row 1 lateral capacity (factored), kips*
$H_{r2} = 11$	Pile row 2 lateral capacity (factored), kips*
$H_{r3} = 14$	Pile row 3 lateral capacity (factored), kips*
$B_{xx} = 12.05$	Pile flange width (normal to wall alignment) dimension, in
$B_{yy} = 11.78$	Pile depth (perpendicular to wall alignment) dimension, in

* Based on LPILE or Broms' Method ϕ=1.0

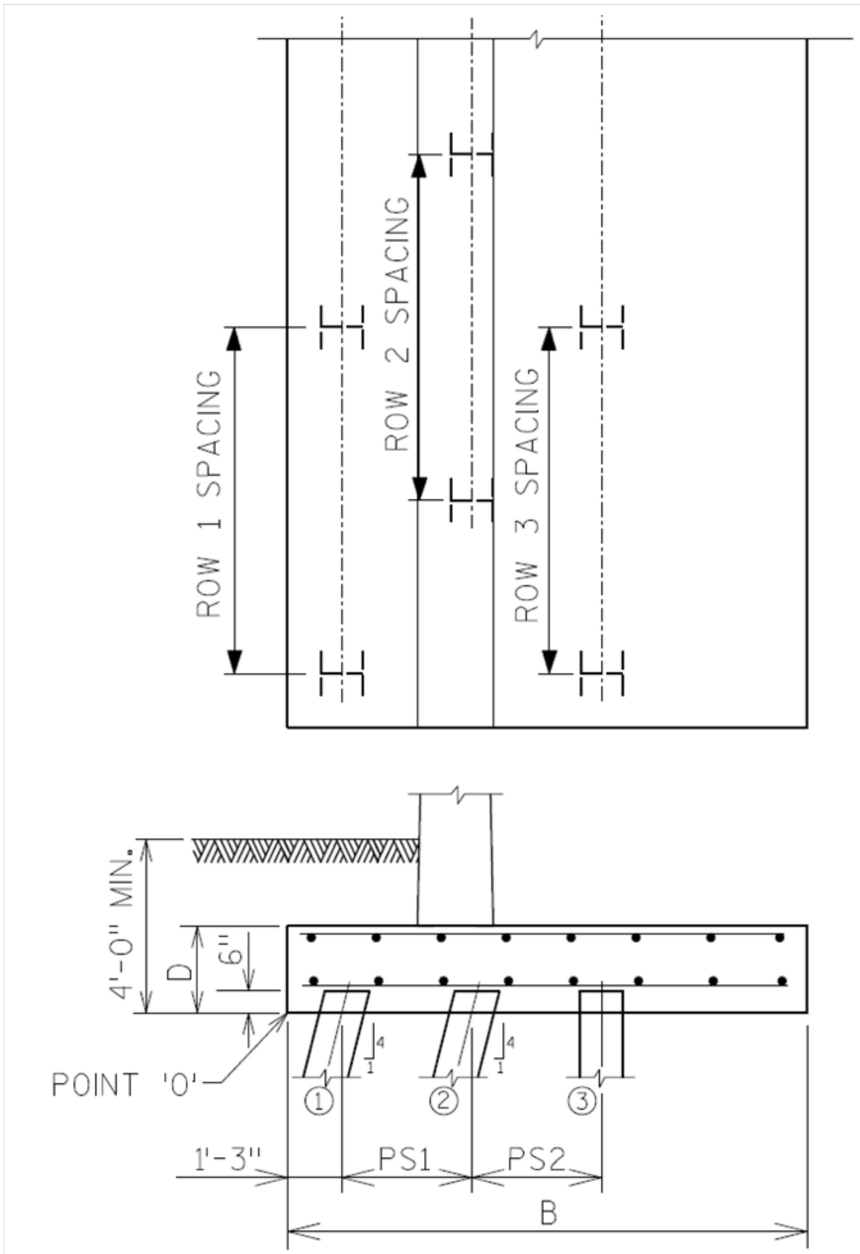

Figure E14-4.3-2
CIP Concrete Pile Geometry

E14-4.4 Permanent and Transient Loads

In this example, load types DC (dead load components), EV (vertical earth pressure), EH (horizontal earth pressure) and LS (live load surcharge) will be used. Passive resistance of the footing will be ignored.

E14-4.4.1 Compute Active Earth Pressure Coefficient

Compute the coefficient of active earth pressure using Coulomb Theory
LRFD [Eq 3.11.5.3-1]

$\phi_f = 32.0 \text{ deg}$

$\beta = 0.0 \text{ deg}$

$\theta = 87.6 \text{ deg}$

$\delta = 17.0 \text{ deg}$

$$k_a = \frac{\sin(\theta + \phi_f)^2}{\Gamma \sin(\theta)^2 \sin(\theta - \delta)}$$

$$\Gamma = \left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2$$

$\boxed{\Gamma = 2.727}$

$$k_a = \frac{\sin(\theta + \phi_f)^2}{\Gamma \sin(\theta)^2 \sin(\theta - \delta)}$$

$\boxed{k_a = 0.294}$

E14-4.4.2 Compute Pile Group Properties

Compute the distance from Point 'O' to the pile row centerlines

$y_{p1} = 1.25$ $\boxed{y_{p1} = 1.25}$ ft

$y_{p2} = y_{p1} + PS1$ $\boxed{y_{p2} = 4.00}$ ft

$y_{p3} = y_{p1} + PS1 + PS2$ $\boxed{y_{p3} = 7.00}$ ft

Compute the effective number of piles in each pile row and overall

$$NP_1 = \begin{cases} \frac{1}{P_1} & \text{if } P_1 > 0 \\ 0 & \text{otherwise} \end{cases}$$ $\boxed{NP_1 = 0.13}$ piles/ft

$$NP_2 = \begin{cases} \frac{1}{P_2} & \text{if } P_2 > 0 \\ 0 & \text{otherwise} \end{cases}$$ $\boxed{NP_2 = 0.13}$ piles/ft

$$NP_3 = \begin{cases} \frac{1}{P_3} & \text{if } P_3 > 0 \\ 0 & \text{otherwise} \end{cases}$$ $\boxed{NP_3 = 0.13}$ piles/ft

$NP = NP_1 + NP_2 + NP_3$ $\boxed{NP = 0.38}$ piles/ft

Compute the centroid of the pile group

$$yy = \begin{cases} \frac{y_{p1}\,NP_1 + y_{p2}\,NP_2 + y_{p3}\,NP_3}{NP} & \text{if } NP > 0 \\ 0 & \text{otherwise} \end{cases}$$ $\boxed{yy = 4.08}$ ft

Compute the distance from the centroid to the pile row

$d_{p1} = yy - y_{p1}$ $\boxed{d_{p1} = 2.83}$ ft

$d_{p2} = yy - y_{p2}$ $\boxed{d_{p2} = 0.08}$ ft

$d_{p3} = yy - y_{p3}$ $\boxed{d_{p3} = -2.92}$ ft

Compute the section modulus for each of the pile rows

$$Sxx_1 = \frac{NP_1\,d_{p1}^2 + NP_2\,d_{p2}^2 + NP_3\,d_{p3}^2}{d_{p1}}$$ $\boxed{Sxx_1 = 0.73}$

$$Sxx_2 = \frac{NP_1\,d_{p1}^2 + NP_2\,d_{p2}^2 + NP_3\,d_{p3}^2}{d_{p2}}$$ $\boxed{Sxx_2 = 24.81}$

$$Sxx_3 = \frac{NP_1\,d_{p1}^2 + NP_2\,d_{p2}^2 + NP_3\,d_{p3}^2}{d_{p3}}$$ $\boxed{Sxx_3 = -0.71}$

E14-4.4.3 Compute Unfactored Loads

The forces and moments are computed by using Figures E14-1.3-1 and E14-1.3-3 and by their respective load types **LRFD [Tables 3.4.1-1 and 3.4.1-2]**

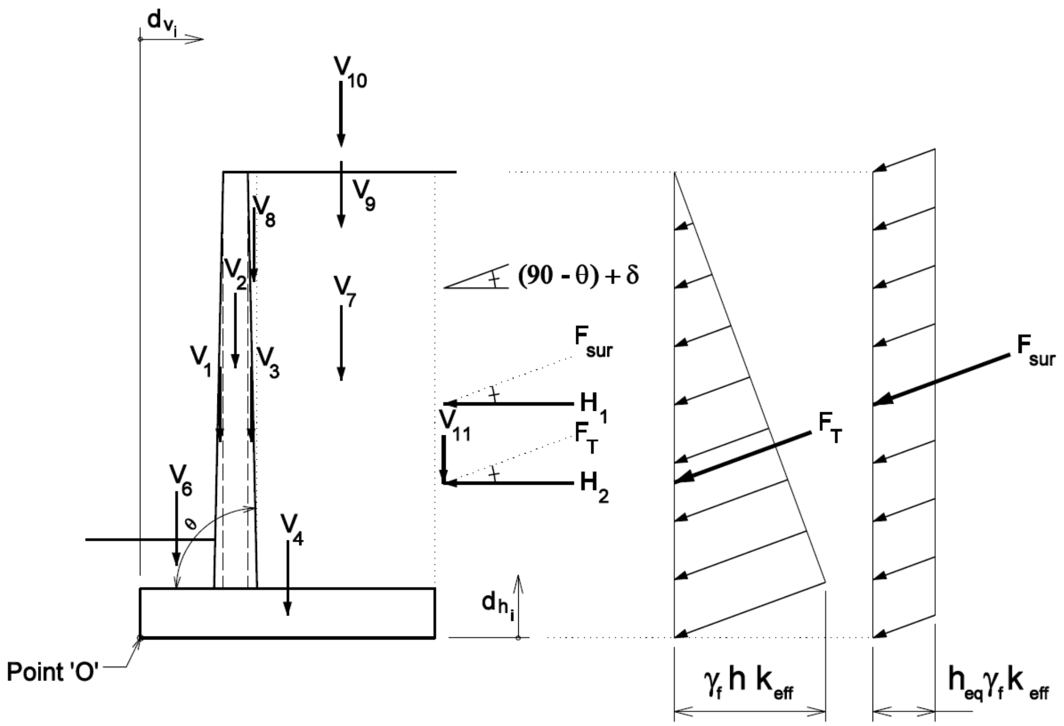

Figure E14-4.4-1
CIP Concrete Wall - External Stability

Active Earth Force Resultant (kip/ft), F_T

$F_T = \frac{1}{2} \gamma_f h^2 k_a$ Active earth force resultant (EH) $\boxed{F_T = 10.17}$

Live Load Surcharge Load (kip/ft), F_{sur}

$F_{sur} = \gamma_f h_{eq} h k_a$ Live load surcharge resultant (LS) $\boxed{F_{sur} = 0.71}$

Vertical Loads (kip/ft), V_i

$V_1 = \frac{1}{2} T_1 h' \gamma_c$ Wall stem front batter (DC) $\boxed{V_1 = 0.72}$

$V_2 = T_t h' \gamma_c$ Wall stem (DC) $\boxed{V_2 = 3.23}$

$V_3 = \frac{1}{2} T_2\ h'\ \gamma_c$ Wall stem back batter (DC) $\boxed{V_3 = 1.44}$

$V_4 = D\ B\ \gamma_c$ Wall footing (DC) $\boxed{V_4 = 4.50}$

$V_6 = A\left(D_f - D\right)\gamma_{fd}$ Soil backfill - toe (EV) $\boxed{V_6 = 0.78}$

$V_7 = C\ h'\ \gamma_f$ Soil backfill - heel (EV) $\boxed{V_7 = 12.66}$

$V_8 = \frac{1}{2}\ T_2\ h'\ \gamma_f$ Soil backfill - batter (EV) $\boxed{V_8 = 1.16}$

$$V_9 = \frac{1}{2}\left(T_2 + C\right)\left[\left(T_2 + C\right)\tan(\beta)\right]\gamma_f$$

Soil backfill - backslope (EV) $\boxed{V_9 = 0.00}$

$$V_{10} = h_{eq}\left(T_2 + C\right)\gamma_f$$

Live load surcharge (LS) $\boxed{V_{10} = 0.58}$

$$V_{11} = F_T\ \sin[(90\ \text{deg} - \theta) + \delta]$$

Active earth force resultant (vertical component - EH) $\boxed{V_{11} = 3.38}$

Moments produced from vertical loads about Point 'O' (kip-ft/ft), MV_i

Moment Arm (ft)		Moment (kip-ft/ft)	
$d_{v1} = A + \frac{2}{3}\ T_1$	$\boxed{d_{v1} = 5.0}$	$MV_1 = V_1\ d_{v1}$	$\boxed{MV_1 = 3.6}$
$d_{v2} = A + T_1 + \frac{T_t}{2}$	$\boxed{d_{v2} = 5.7}$	$MV_2 = V_2\ d_{v2}$	$\boxed{MV_2 = 18.4}$
$d_{v3} = A + T_1 + T_t + \frac{T_2}{3}$	$\boxed{d_{v3} = 6.5}$	$MV_3 = V_3\ d_{v3}$	$\boxed{MV_3 = 9.4}$
$d_{v4} = \frac{B}{2}$	$\boxed{d_{v4} = 6.0}$	$MV_4 = V_4\ d_{v4}$	$\boxed{MV_4 = 27.0}$
$d_{v6} = \frac{A}{2}$	$\boxed{d_{v6} = 2.4}$	$MV_6 = V_6\ d_{v6}$	$\boxed{MV_6 = 1.9}$

$$d_{v7} = B - \frac{C}{2} \qquad \boxed{d_{v7} = 9.5} \qquad MV_7 = V_7\, d_{v7} \qquad \boxed{MV_7 = 120.8}$$

$$d_{v8} = A + T_1 + T_t + \frac{2T_2}{3} \qquad \boxed{d_{v8} = 6.8} \qquad MV_8 = V_8\, d_{v8} \qquad \boxed{MV_8 = 7.9}$$

$$d_{v9} = A + T_1 + T_t + \frac{2\left(T_2 + C\right)}{3} \qquad \boxed{d_{v9} = 10.1} \qquad MV_9 = V_9\, d_{v9} \qquad \boxed{MV_9 = 0.0}$$

$$d_{v10} = B - \left(\frac{T_2 + C}{2}\right) \qquad \boxed{d_{v10} = 9.1} \qquad MV_{10} = V_{10}\, d_{v10} \qquad \boxed{MV_{10} = 5.3}$$

$$d_{v11} = B \qquad \boxed{d_{v11} = 12.0} \qquad MV_{11} = V_{11}\, d_{v11} \qquad \boxed{MV_{11} = 40.5}$$

Horizontal Loads (kip/ft), H_i

$H_1 = F_{sur} \cos[(90\ deg - \theta) + \delta]$ Live load surcharge (LS) $\boxed{H_1 = 0.67}$

$H_2 = F_T \cos[(90\ deg - \theta) + \delta]$ Active earth force (horizontal component) (EH) $\boxed{H_2 = 9.59}$

Moments produced from horizontal loads about about Point 'O' (kip-ft/ft), MH_i

Moment Arm (ft)		Moment (kip-ft/ft)	
$d_{h1} = \frac{h}{2}$	$\boxed{d_{h1} = 12.0}$	$MH_1 = H_1\, d_{h1}$	$\boxed{MH_1 = 8.0}$
$d_{h2} = \frac{h}{3}$	$\boxed{d_{h2} = 8.0}$	$MH_2 = H_2\, d_{h2}$	$\boxed{MH_2 = 76.8}$

Summary of Unfactored Forces & Moments:

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
V_1	Wall stem front batter	0.72	d_{v1}	5.0	MV_1	3.6	DC
V_2	Wall stem	3.23	d_{v2}	5.7	MV_2	18.4	DC
V_3	Wall stem back batter	1.44	d_{v3}	6.5	MV_3	9.4	DC
V_4	Wall footing	4.50	d_{v4}	6.0	MV_4	27.0	DC
V_6	Soil backfill - Toe	0.78	d_{v6}	2.4	MV_6	1.9	EV
V_7	Soil backfill - Heel	12.66	d_{v7}	9.5	MV_7	120.8	EV
V_8	Soil backfill - Batter	1.16	d_{v8}	6.8	MV_8	7.9	EV
V_9	Soil backfill - Backslope	0.00	d_{v9}	10.1	MV_9	0.0	EV
V_{10}	Live load surcharge	0.58	d_{v10}	9.1	MV_{10}	5.3	LS
V_{11}	Active earth pressure	3.38	d_{v11}	12.0	MV_{11}	40.5	EH

Table E14-4.4-1
Unfactored Vertical Forces & Moments

Load			Moment Arm		Moment		LRFD Load Type
Item	Description	Value (kip/ft)	Item	Value (ft)	Item	Value (kip-ft/ft)	
H_1	Live load surcharge	0.67	d_{h1}	12.0	MH_1	8.0	LS
H_2	Active earth force	9.59	d_{h2}	8.0	MH_2	76.8	EH

Table E14-4.4-2
Unfactored Horizontal Forces & Moments

E14-4.4.4 Summarize Applicable Load and Resistance Factors

Maximum and minimum load factors shall be used to determine the extreme load effects. WisDOT's policy is to set all the load modifiers to zero($n = 1.0$). Factored loads and moments for each limit state are calculated by applying the appropriate load factors **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The following load combinations will be used in this example:

Load Combination	γ_{DC}	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	Application
Strength Ia	0.90	1.00	-	1.75	1.50	Sliding, Eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	Bearing, Wall Strength
Service I	1.00	1.00	1.00	1.00	1.00	Wall Crack Control

Table E14-4.4-3
Load Combinations

Load Combination Assumptions:

- Live load surcharge stabilizing loads (if applicable) are ignored for overturning and sliding analyses. Live load surcharge is used to compute maximum bearing pressure, wall strength and overall (global) stability.
- Minimum horizontal earth pressure ,$\gamma_{EH(min)}$ = 0.9, will not control in this example based on B/H and lateral load inclination, but should be checked.
- Component load factors shall remain consistent throughout calculations. For example, the active earth force resultant (F_T) can be broken into component forces of either $V_{10}\gamma_{EH(max)}$ and $H_2\gamma_{EH(max)}$ or $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(min)}$, not $V_{10}\gamma_{EH(min)}$ and $H_2\gamma_{EH(max)}$.

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states for each design check.

E14-4.4.5 Compute Factored Loads and Moments

Unfactored loads by load type (kip/ft)

$$V_{DC} = V_1 + V_2 + V_3 + V_4 \qquad \boxed{V_{DC} = 9.9}$$

$$V_{EV} = V_6 + V_7 + V_8 + V_9 \qquad \boxed{V_{EV} = 14.6}$$

$$V_{LS} = V_{10} \qquad \boxed{V_{LS} = 0.6}$$

$$V_{EH} = V_{11} \qquad \boxed{V_{EH} = 3.4}$$

$$H_{LS} = H_1 \qquad \boxed{H_{LS} = 0.7}$$

$$H_{EH} = H_2 \qquad \boxed{H_{EH} = 9.6}$$

Unfactored moments by load type (kip-ft/ft)

$$M_{DC} = MV_1 + MV_2 + MV_3 + MV_4 \qquad \boxed{M_{DC} = 58.4}$$

$$M_{EV} = MV_6 + MV_7 + MV_8 + MV_9 \qquad \boxed{M_{EV} = 130.6}$$

$$M_{LS1} = MV_{10} \qquad \boxed{M_{LS1} = 5.3}$$

$$M_{EH1} = MV_{11} \qquad \boxed{M_{EH1} = 40.5}$$

$$M_{LS2} = MH_1 \qquad \boxed{M_{LS2} = 8.0}$$

$$M_{EH2} = MH_2 \qquad \boxed{M_{EH2} = 76.8}$$

Factored vertical loads by limit state (kip/ft)

$$V\_Ia = n\left(0.90V_{DC} + 1.00V_{EV} + 0.00\ V_{LS} + 1.50\ V_{EH}\right) \qquad \boxed{V\_Ia = 28.6}$$

$$V\_Ib = n\left(1.25V_{DC} + 1.35V_{EV} + 1.75\ V_{LS} + 1.50\ V_{EH}\right) \qquad \boxed{V\_Ib = 38.2}$$

$$V\_Ser = n\left(1.00V_{DC} + 1.00V_{EV} + 1.00\ V_{LS} + 1.00\ V_{EH}\right) \qquad \boxed{V\_Ser = 28.4}$$

Factored horizontal loads by limit state (kip/ft)

$$H\_Ia = n\left(1.75H_{LS} + 1.50H_{EH}\right) \qquad \boxed{H\_Ia = 15.6}$$

$$H\_Ib = n\left(1.75H_{LS} + 1.50H_{EH}\right) \qquad \boxed{H\_Ib = 15.6}$$

$$H\_Ser = n\left(1.00H_{LS} + 1.00H_{EH}\right) \qquad \boxed{H\_Ser = 10.3}$$

Factored moments produced by vertical Loads by limit state (kip-ft/ft)

$$MV\_Ia = n\left(0.90M_{DC} + 1.00M_{EV} + 0.00M_{LS1} + 1.50\ M_{EH1}\right) \qquad \boxed{MV\_Ia = 243.9}$$

$$MV\_Ib = n\left(1.25M_{DC} + 1.35M_{EV} + 1.75M_{LS1} + 1.50\ M_{EH1}\right) \qquad \boxed{MV\_Ib = 319.3}$$

$$MV\_Ser = n\left(1.00M_{DC} + 1.00M_{EV} + 1.00M_{LS1} + 1.00\ M_{EH1}\right) \qquad \boxed{MV\_Ser = 234.8}$$

Factored moments produced by horizontal loads by limit state (kip-ft/ft)

$$MH\_Ia = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right) \qquad \boxed{MH\_Ia = 129.1}$$

$$MH\_Ib = n\left(1.75M_{LS2} + 1.50\ M_{EH2}\right) \qquad \boxed{MH\_Ib = 129.1}$$

$$MH\_Ser = n\left(1.00M_{LS2} + 1.00\ M_{EH2}\right) \qquad \boxed{MH\_Ser = 84.8}$$

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	28.6	243.9	15.6	129.1
Strength Ib	38.2	319.3	15.6	129.1
Service I	28.4	234.8	10.3	84.8

Table E14-4.4-4
Summary of Factored Loads & Moments

E14-4.5 Evaluate Pile Reactions

Calculated loads for each limit state:

Strength Ia	Strength Ib	Service	
$\boxed{V\_Ia = 28.56}$	$\boxed{V\_Ib = 38.15}$	$\boxed{V\_Ser = 28.45}$	Vertical Load, kip/ft
$\boxed{H\_Ia = 15.56}$	$\boxed{H\_Ib = 15.56}$	$\boxed{H\_Ser = 10.26}$	Horizontal Load, kip/ft
$\boxed{MV\_Ia = 243.90}$	$\boxed{MV\_Ib = 319.27}$	$\boxed{MV\_Ser = 234.76}$	Moments (Vertical) kip-ft/ft
$\boxed{MH\_Ia = 129.13}$	$\boxed{MH\_Ib = 129.13}$	$\boxed{MH\_Ser = 84.75}$	Moments (Horizontal), kip-ft/ft

Compute the eccentricity about Point 'O'

$$e_{toe\_Ia} = \frac{MH\_Ia - MV\_Ia}{V\_Ia}$$ Strength Ia $\boxed{e_{toe\_Ia} = -4.02}$ ft

$$e_{toe\_Ib} = \frac{MH\_Ib - MV\_Ib}{V\_Ib}$$ Strength Ib $\boxed{e_{toe\_Ib} = -4.98}$ ft

$$e_{toe\_Ser} = \frac{MH\_Ser - MV\_Ser}{V\_Ser}$$ Service $\boxed{e_{toe\_Ser} = -5.27}$ ft

Compute the eccentricity about the neutral axis of the pile group

$e_{NA\_Ia} = yy + e_{toe\_Ia}$ Strength Ia $\boxed{e_{NA\_Ia} = 0.07}$ ft

$e_{NA\_Ib} = yy + e_{toe\_Ib}$ Strength Ib $\boxed{e_{NA\_Ib} = -0.90}$ ft

$e_{NA\_Ser} = yy + e_{toe\_Ser}$ Service $\boxed{e_{NA\_Ser} = -1.19}$ ft

Compute the moment about the neutral axis of the pile group

$M_{NA\_Ia} = V\_Ia \; e_{NA\_Ia}$ Strength Ia $\boxed{M_{NA\_Ia} = 1.9}$ kip-ft/ft

$M_{NA\_Ib} = V\_Ib \; e_{NA\_Ib}$ Strength Ib $\boxed{M_{NA\_Ib} = -34.4}$ kip-ft/ft

$M_{NA\_Ser} = V\_Ser \; e_{NA\_Ser}$ Service $\boxed{M_{NA\_Ser} = -33.9}$ kip-ft/ft

Compute the pile reactions for each limit state

Strength Ia

$$P_{U1a} = \frac{V\_Ia}{NP} + \frac{M_{NA\_Ia}}{Sxx_1}$$ $\boxed{P_{U1a} = 78.7}$ kip/pile

$$P_{U2a} = \frac{V\_Ia}{NP} + \frac{M_{NA\_Ia}}{Sxx_2}$$ $\boxed{P_{U2a} = 76.2}$ kip/pile

$$P_{U3a} = \frac{V\_Ia}{NP} + \frac{M_{NA\_Ia}}{Sxx_3}$$ $\boxed{P_{U3a} = 73.5}$ kip/pile

Strength Ib

$$P_{U1b} = \frac{V\_Ib}{NP} + \frac{M_{NA\_Ib}}{Sxx_1}$$ $\boxed{P_{U1b} = 54.6}$ kip/pile

$$P_{U2b} = \frac{V\_Ib}{NP} + \frac{M_{NA\_Ib}}{Sxx_2}$$ $\boxed{P_{U2b} = 100.4}$ kip/pile

$$P_{U3b} = \frac{V\_Ib}{NP} + \frac{M_{NA\_Ib}}{Sxx_3}$$ $\boxed{P_{U3b} = 150.2}$ kip/pile

Service

$$P_{U1\_Ser} = \frac{V\_Ser}{NP} + \frac{M_{NA\_Ser}}{Sxx_1}$$ $\boxed{P_{U1\_Ser} = 29.5}$ kip/pile

$$P_{U2\_Ser} = \frac{V\_Ser}{NP} + \frac{M_{NA\_Ser}}{Sxx_2}$$ $\boxed{P_{U2\_Ser} = 74.5}$ kip/pile

$$P_{U3\_Ser} = \frac{V\_Ser}{NP} + \frac{M_{NA\_Ser}}{Sxx_3}$$ $\boxed{P_{U3\_Ser} = 123.6}$ kip/pile

Load Combination	Row 1 (kip/pile)	Row 2 (kip/pile)	Row 3 (kip/pile)
Strength Ia	78.7	76.2	73.5
Strength Ib	54.6	100.4	150.2
Service I	29.5	74.5	123.6

Table E14-4.5-1
Summary of Factored Pile Reactions (Vertical)

E14-4.6 Evaluate External Stability of Wall

Three potential external failure mechanisms will be considered in this example. These failures include pile bearing resistance, limiting eccentricity and lateral resistance. Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-4.6.1 Pile Bearing Resistance

Axial and lateral pile capacities from Geotechnical Site Investigation Report:

$Pile\_Axial = 220$ Pile axial capacity, kips

$pile\_batter = 4$ Pile batter (pile_batter V:1H)

$H_{r1} = 11.00$ Battered pile row 1 lateral capacity, kips/pile

$H_{r2} = 11.00$ Battered pile row 2 lateral capacity, kips/pile

$H_{r3} = 14.00$ Vertical pile row 3 lateral capacity, kips/pile

Determine the horizontal and vertical components of the battered pile

$$pile\_angle = atan\left(\frac{1}{pile\_batter}\right) \qquad \boxed{pile\_angle = 14.0\ deg}$$

$$P_{Rb\_H} = Pile\_Axial\ \sin(pile\_angle) \qquad \boxed{P_{Rb\_H} = 53.4} \text{ kips/pile}$$

$$P_{Rb\_V} = Pile\_Axial\ \cos(pile\_angle) \qquad \boxed{P_{Rb\_V} = 213.4} \text{ kips/pile}$$

Calculate axial capacity of battered piles

$$P_R = P_{Rb\_V} \qquad \boxed{P_R = 213.4} \text{ kips/pile}$$

$$P_u = \max(P_{U1a}, P_{U2a}, P_{U1b}, P_{U2b}) \qquad \boxed{P_u = 100.4} \text{ kips/pile}$$

$$CDR_{Brg\_B\_Pile} = \frac{P_R}{P_u} \qquad \boxed{CDR_{Brg\_B\_Pile} = 2.13}$$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

Calculate axial capacity of vertical piles

$$P_R = Pile\_Axial \qquad \boxed{P_R = 220.0}$$

$$P_u = \max(P_{U3a}, P_{U3b}) \qquad \boxed{P_u = 150.2}$$

$$CDR_{Brg\_V\_Pile} = \frac{P_R}{P_u} \qquad \boxed{CDR_{Brg\_V\_Pile} = 1.46}$$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-4.6.2 Pile Sliding Resistance

For sliding failure, the horizontal force effects, H_u, is checked against the sliding resistance, H_R, where $H_R = \phi H_n$. The following calculations are based on **Strength Ia**:

Factored Lateral Force, H_u

$H_u = H\_Ia$ $\boxed{H_u = 15.6}$ kip/ft

Sliding Resistance, H_R

It is assumed that the P-y method was used for the pile analysis (LPILE), thus group effects shall be considered. Calculate sliding capacity of the effective pile group per **LRFD [Table-10.7.2.4-1]:**

$B_{yy} = 11.78$ Depth of pile, in

$$\frac{PS1 + PS2}{\frac{B_{yy}}{12}} = 5.86$$ Say:5B

Note: It was assumed that pile row 1 and 3 are aligned throughout the pile group and that pile row 2 will not effect the lateral pile group resistance. Pile row 1 and 3 will then be applied row 1 and 2 "5B" multipliers, respectfully.

"5B" Pile multipliers

$row1 = 1.00$

$row2 = 1.00$

$row3 = 0.80$

Lateral group resistance

$H_{R1} = row1\ H_{r1}\ NP_1 + row2\ H_{r2}\ NP_2 + row3\ H_{r3}\ NP_3$ $\boxed{H_{R1} = 4.15}$ kip/ft

Batter resistance

$H_{R2} = P_{Rb\_H}\left(NP_1 + NP_2\right)$ $\boxed{H_{R2} = 13.34}$ kip/ft

Compute factored resistance against failure by sliding, R_R

$H_R = H_{R1} + H_{R2}$ $\boxed{H_R = 17.49}$ kip/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} = \frac{H_R}{H_u}$$ $\boxed{CDR_{Sliding} = 1.12}$

Is the CDR ≥ 1.0 ? $\boxed{check = \text{"OK"}}$

E14-4.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. Crack control and temperature and shrinkage considerations will also be included.

E14-4.7.1 Evaluate Wall Footing

Investigate shear and moment requirements

E14-4.7.1.1 Evalute One-Way Shear

Design for one-way shear in only the transverse direction.

Compute the effective shear depth,d_v, for the heel:

$cover = 2.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 7$ (transverse bar size)

$Bar_D = 0.875$ in (transverse bar diameter)

$Bar_A = 0.600$ in^2 (transverse bar area)

$$A_{s\_heel} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_heel} = 0.80} \ in^2/ft$$

$$d_{s\_heel} = D \cdot 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_heel} = 27.6} \ in$$

$\alpha_1 = 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$$a\_heel = \frac{A_{s\_heel} f_y}{\alpha_1 f'_c b} \qquad \boxed{a\_heel = 1.3} \ in$$

$$d_{v1} = d_{s\_heel} - \frac{a\_heel}{2} \qquad \boxed{d_{v1} = 26.9} \ in$$

$$d_{v2} = 0.9 \, d_{s\_heel} \qquad \boxed{d_{v2} = 24.8} \ in$$

$$d_{v3} = 0.72 \, D \, 12 \qquad \boxed{d_{v3} = 21.6} \ in$$

$$d_{v\_heel} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_heel} = 26.9} \ in$$

Compute the effective shear depth,d_v, for the toe

$cover = 6.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 7$ (transverse bar size)

$Bar_D = 0.88$ in (transverse bar diameter)

$Bar_A = 0.60$ in^2 (transverse bar area)

$$A_{s\_toe} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_toe} = 0.80} \text{ in}^2/\text{ft}$$

$$d_{s\_toe} = D\ 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_toe} = 23.6} \text{ in}$$

$$a\_toe = \frac{A_{s\_toe}\ f_y}{\alpha_1\ f'_c\ b} \qquad \boxed{a\_toe = 1.3} \text{ in}$$

$$d_{v1} = d_{s\_toe} - \frac{a\_toe}{2} \qquad \boxed{d_{v1} = 22.9} \text{ in}$$

$$d_{v2} = 0.9\ d_{s\_toe} \qquad \boxed{d_{v2} = 21.2} \text{ in}$$

$$d_{v\_toe} = \max(d_{v1}, d_{v2}) \qquad \boxed{d_{v\_toe} = 22.9} \text{ in}$$

Determine the distance from Point 'O' to the critical sections:

$$y\_crit\_toe = A\ 12 - d_{v\_toe} \qquad \boxed{y\_crit\_toe = 34.1} \text{ in}$$

$$y\_crit\_heel = B\ 12 - C\ 12 + d_{v\_heel} \qquad \boxed{y\_crit\_heel = 112.0} \text{ in}$$

Determine the distance from Point 'O' to the pile limits:

$$y_{v1\_neg} = y_{p1}\ 12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_neg} = 9.1} \text{ in}$$

$$y_{v1\_pos} = y_{p1}\ 12 + \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_pos} = 20.9} \text{ in}$$

$$y_{v2\_neg} = y_{p2}\ 12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v2\_neg} = 42.1} \text{ in}$$

$$y_{v2\_pos} = y_{p2}\,12 + \frac{B_{yy}}{2}$$ $\boxed{y_{v2\_pos} = 53.9}$ in

$$y_{v3\_neg} = y_{p3}\,12 - \frac{B_{yy}}{2}$$ $\boxed{y_{v3\_neg} = 78.1}$ in

$$y_{v3\_pos} = y_{p3}\,12 + \frac{B_{yy}}{2}$$ $\boxed{y_{v3\_pos} = 89.9}$ in

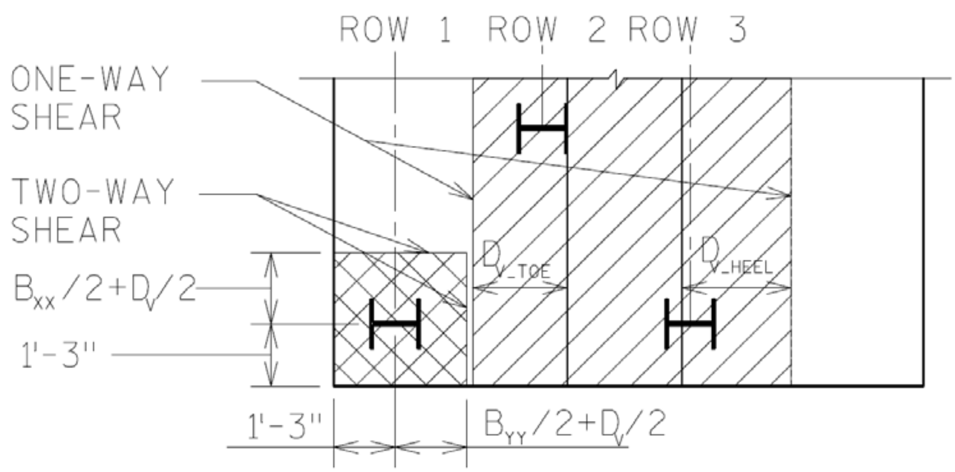

Figure E14-4.7-1
Partial Footing Plan for Critical Shear Sections

Determine if the pile rows are "Outside", "On", or "Inside" the critical sections

Since the pile row 1 falls "Outside" the critical sections, the full row pile reaction will be used for shear

$$P_{U1} = \max\left(P_{U1a}, P_{U1b}\right)$$ $\boxed{P_{U1} = 78.7}$ kip

$$V_{u\_Pile1} = 1.0\left(P_{U1}\,NP_1\right)$$ $\boxed{V_{u\_Pile1} = 9.8}$ kip/ft

Since the pile row 2 and 3 falls "Inside" the critical sections, none of the row pile reactions will be used for shear

The load applied to the critical section is based on the proportion of the piles located outside of the critical toe or heel section. In this case, pile row 1 falls outside the toe critical section and the full row pile reaction will be used for shear.

$V_u = V_{u\_Pile1}$ $\boxed{V_u = 9.8}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\ \beta\lambda\sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Nominal one-way action shear resistance for structures without transverse reinforcement, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_c = 0.0316\ \beta\ \lambda\sqrt{f'_c}\ b\ d_{v\_toe}$ $\boxed{V_c = 32.5}$ kip/ft

$V_{n1} = V_c$ $\boxed{V_{n1} = 32.5}$ kip/ft

$V_{n2} = 0.25\ f'_c\ b\ d_{v\_toe}$ $\boxed{V_{n2} = 240.3}$ kip/ft

$V_n = \min(V_{n1}, V_{n2})$ $\boxed{V_n = 32.5}$ kip/ft

$\phi_v = 0.90$

$V_r = \phi_v\ V_n$ $\boxed{V_r = 29.2}$ kip/ft

$\boxed{V_u = 9.8}$ kip/ft

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-4.7.1.2 Evaluate Two-Way Shear

For two-way action around the maximum loaded pile, the pile critical perimeter, b_o, is located a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge.

Two-way action should be checked for the maximum loaded pile.

$V_u = \max(P_{U1a}, P_{U2a}, P_{U3a}, P_{U1b}, P_{U2b}, P_{U3b})$ $\boxed{V_u = 150.2}$ kip

Determine the location of the pile critical perimeter. Assume that the critical section is outside of the footing and only include the portion of the shear perimeter is located within the footing:

$$b_{o\_xx} = 1.25\,12 + \frac{B_{xx}}{2} + \frac{d_{v\_toe}}{2}$$ $\boxed{b_{o\_xx} = 32.5}$ in

$$b_{o\_yy} = 1.25\,12 + \frac{B_{yy}}{2} + \frac{d_{v\_toe}}{2}$$ $\boxed{b_{o\_yy} = 32.3}$ in

$$\beta_{c\_pile} = \frac{b_{o\_xx}}{b_{o\_yy}}$$ $\boxed{\beta_{c\_pile} = 1.004}$ in

$$b_{o\_pile} = b_{o\_xx} + b_{o\_yy}$$ $\boxed{b_{o\_pile} = 64.8}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.13.3.6.3]**

$\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_{n1} = \left(0.063 + \frac{0.126}{\beta_{c\_pile}}\right)\lambda\sqrt{f'_c}\,b_{o\_pile}\,d_{v\_toe}$$ $\boxed{V_{n1} = 523.1}$ kip/ft

$$V_{n2} = 0.126\,\lambda\sqrt{f'_c}\,b_{o\_pile}\,d_{v\_toe}$$ $\boxed{V_{n2} = 349.7}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 349.7}$ kip/ft

$$V_r = \phi_v\,V_n$$ $\boxed{V_r = 314.7}$ kip/ft

$\boxed{V_u = 150.2}$ kip/ft

Is V_u less than V_r? $\boxed{check = "OK"}$

E14-4.7.1.3 Evaluate Top Transverse Reinforcement Strength

Top transverse reinforcement strength is determined by assuming the heel acts as a cantilever member supporting its own weight and loads acting above it. Pile reactions may be used to decrease this load.

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B}V_4\right) + 1.35\,(V_7 + V_8 + V_9) + 1.75\,(V_{10}) + 1.50\,(V_{11})$$

$\boxed{V_u = 27.0}$ kip/ft

$$M_u = V_u\,\frac{C}{2}$$ $\boxed{M_u = 66.3}$ kip-ft/ft

Calculated the capacity of the heel in flexure at the face of the stem:

$$M_n = A_{s\_heel}\,f_y\left(d_{s\_heel} - \frac{a\_heel}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 107.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \dfrac{a\_heel}{\beta_1}$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_{s\_heel}}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_{s\_heel}}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_{s\_heel}}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Note:	if $\phi_F = 0.75$	Section is compression-controlled
	if $0.75 < \phi_F < 0.90$	Section is in transition
	if $\phi_F = 0.90$	Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\, M_n$ $\boxed{M_r = 96.8}$ kip-ft/ft

$\boxed{M_u = 66.3}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\,\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.449}$ ksi

$I_g = \dfrac{1}{12}\, b\, (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \dfrac{1}{2}\, D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = \gamma_3(\gamma_1\, f_r)\, S_c$ therefore, $M_{cr} = 1.1\, f_r\, S_c$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$$ $\boxed{M_{cr} = 74.1}$ kip-ft/ft

$\boxed{1.33\, M_u = 88.2}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33{*}M_u$? $\boxed{check = \text{"OK"}}$

E14-4.7.1.4 Evaluate Bottom Transverse Reinforcement Strength

Bottom transverse reinforcement strength is determined by using the maximum pile reaction.

Determine the moment arms

$$arm\_v1 = A - y_{p1}$$ $\boxed{arm\_v1 = 3.5}$ ft

$$arm\_v2 = A - y_{p2}$$ $\boxed{arm\_v2 = 0.8}$ ft

Determine the moment for **Strength Ia**:

$$V_{u\_1a} = P_{U1a}\, NP_1$$ $\boxed{V_{u\_1a} = 9.8}$ kip/ft

$$V_{u\_2a} = P_{U2a}\, NP_2$$ $\boxed{V_{u\_2a} = 9.5}$ kip/ft

$$M_{u\_Ia} = V_{u\_1a}\, arm\_v1 + V_{u\_2a}\, arm\_v2$$ $\boxed{M_{u\_Ia} = 41.6}$ kip-ft/ft

Determine the moment for **Strength Ib**:

$$V_{u\_1b} = P_{U1b}\, NP_1$$ $\boxed{V_{u\_1b} = 6.8}$ kip/ft

$$V_{u\_2b} = P_{U2b}\, NP_2$$ $\boxed{V_{u\_2b} = 12.5}$ kip/ft

$$M_{u\_Ib} = V_{u\_1b}\, arm\_v1 + V_{u\_2b}\, arm\_v2$$ $\boxed{M_{u\_Ib} = 33.3}$ kip-ft/ft

Determine the design moment:

$$M_u = \max\left(M_{u\_Ia}, M_{u\_Ib}\right)$$ $\boxed{M_u = 41.6}$ kip-ft/ft

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_{s\_toe}\, f_y\left(d_{s\_toe} - \frac{a\_toe}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 91.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_{s\_toe}}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_{s\_toe}}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_{s\_toe}}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\ M_n$ $\boxed{M_r = 82.4}$ kip-ft/ft

$\boxed{M_u = 41.6}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\ \lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.449}$ ksi

$I_g = \frac{1}{12}\ b\ (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \frac{1}{2}\ D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \frac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = 1.1\ f_r\ S_c\ \frac{1}{12}$ from E14-4.7.1.3 $\boxed{M_{cr} = 74.1}$ kip-ft/ft

$\boxed{1.33\ M_u = 55.3}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$? $\boxed{\text{check} = \text{"OK"}}$

E14-4.7.1.5 Evaluate Longitudinal Reinforcement Strength

The structural design of the longitudinal reinforcement, assuming the footing acts as a continuous beam over pile supports, is calculated using the maximum pile reactions.

Compute the effective shear depth, d_v ,for the longitudinal reinforcement

$cover = 6.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 5$ (longitudinal bar size)

$Bar_D = 0.625$ in (longitudinal bar diameter)

$Bar_A = 0.310$ in^2 (longitudinal bar area)

$$A_{s\_long} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_long} = 0.31} \text{ in}^2\text{/ft}$$

$$d_s = D\ 12 - cover - Bar_{D\_toe} - \frac{Bar_D}{2} \qquad \boxed{d_s = 22.8} \text{ in}$$

$$a\_long = \frac{A_{s\_long}\ f_y}{\alpha_1\ f'_c\ b} \qquad \boxed{a\_long = 0.5} \text{ in}$$

$$d_{v1} = d_s - \frac{a\_long}{2} \qquad \boxed{d_{v1} = 22.6} \text{ in}$$

$$d_{v2} = 0.9\ d_s \qquad \boxed{d_{v2} = 20.5} \text{ in}$$

$$d_{v3} = 0.72\ D\ 12 \qquad \boxed{d_{v3} = 21.6} \text{ in}$$

$$d_{v\_long} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_long} = 22.6} \text{ in}$$

Calculate the design moment using a uniform vertical load:

$$L_{pile} = \max(P_1, P_2, P_3) \qquad \boxed{L_{pile} = 8.0} \text{ ft}$$

$$w_u = \frac{V\_Ib}{B} \qquad \boxed{w_u = 3.2} \text{ kip/ft/ft}$$

$$M_u = \frac{w_u\ L_{pile}^2}{10} \qquad \boxed{M_u = 20.3} \text{ kip-ft/ft}$$

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_{s\_long} f_y \left(d_s - \frac{a\_long}{2}\right)\frac{1}{12}$$ $\boxed{M_n = 35.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$ $\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F M_n$$ $\boxed{M_r = 31.5}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\,\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.449}$ ksi

$$I_g = \frac{1}{12} b (D\ 12)^3$$ $\boxed{I_g = 27000}$ in^4

$$y_t = \frac{1}{2} D\ 12$$ $\boxed{y_t = 15.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1800}$ in^3

$$M_{cr} = 1.1\, f_r\, S_c \frac{1}{12}$$ from E14-4.7.1.3 $\boxed{M_{cr} = 74.1}$ kip-ft/ft

$\boxed{1.33\ M_u = 27.1}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

E14-4.7.2 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$H_1 = \gamma_f h_{eq} h' k_a \cos(90 \deg - \theta + \delta)$ $\boxed{H_1 = 0.6}$ kip/ft

$H_2 = \frac{1}{2} \gamma_f h'^2 k_a \cos(90 \deg - \theta + \delta)$ $\boxed{H_2 = 7.7}$ kip/ft

$M_1 = H_1 \left(\frac{h'}{2}\right)$ $\boxed{M_1 = 6.4}$ kip-ft/ft

$M_2 = H_2 \left(\frac{h'}{3}\right)$ $\boxed{M_2 = 55.2}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$H_{u1} = 1.75 H_1 + 1.50 H_2$ $\boxed{H_{u1} = 12.6}$ kip/ft

$M_{u1} = 1.75 M_1 + 1.50 M_2$ $\boxed{M_{u1} = 94.0}$ kip-ft/ft

for **Service I**:

$H_{u3} = 1.00 H_1 + 1.00 H_2$ $\boxed{H_{u3} = 8.3}$ kip/ft

$M_{u3} = 1.00 M_1 + 1.00 M_2$ $\boxed{M_{u3} = 61.6}$ kip-ft/ft

E14-4.7.2.1 Evaluate Stem Shear Strength at Footing

$V_u = H_{u1}$ $\boxed{V_u = 12.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316 \, \beta \lambda \sqrt{f'_c} \, b_v \, d_v$

$V_{n2} = 0.25 f'_c b_v d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 9$ (transverse bar size)

$Bar_D = 1.13$ in (transverse bar diameter)

$Bar_A = 1.00$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 1.00} \; in^2/ft$$

$$d_s = T_b\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 25.6} \; in$$

$$a = \frac{A_s\,f_y}{\alpha_1\,f'_c\,b} \qquad \boxed{a = 1.7} \; in$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 24.7} \; in$$

$$d_{v2} = 0.9\,d_s \qquad \boxed{d_{v2} = 23.0} \; in$$

$$d_{v3} = 0.72\,T_b\,12 \qquad \boxed{d_{v3} = 20.3} \; in$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 24.7} \; in$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_c = 0.0316\,\beta\,\lambda\sqrt{f'_c}\,b\,d_v \qquad \boxed{V_c = 35.1} \; kip/ft$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 35.1} \; kip/ft$$

$$V_{n2} = 0.25\,f'_c\,b\,d_v \qquad \boxed{V_{n2} = 259.6} \; kip/ft$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 35.1} \; kip/ft$$

$$V_r = \phi_v\,V_n \qquad \boxed{V_r = 31.6} \; kip/ft$$

$$\boxed{V_u = 12.6} \; kip/ft$$

Is V_u less than V_r? $\boxed{check = "OK"}$

E14-4.7.2.2 Evaluate Stem Flexural Strength at Footing

$$M_u = M_{u1} \qquad \boxed{M_u = 94.0} \; kip\text{-}ft/ft$$

Calculate the capacity of the stem in flexure at the face of the footing:

$$M_n = A_s\,f_y\left(d_s - \frac{a}{2}\right)\frac{1}{12} \qquad \boxed{M_n = 123.6} \; kip\text{-}ft/ft$$

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \dfrac{a}{\beta_1}$ $\boxed{c = 1.98}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi,
LRFD [5.5.4.2.1], [Table C5.7.2.1-1]

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 111.2}$ kip-ft/ft

$\boxed{M_u = 94.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\,\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.45}$ ksi

$I_g = \dfrac{1}{12}\, b\, (T_b\, 12)^3$ $\boxed{I_g = 22247}$ in^4

$y_t = \dfrac{1}{2}\, T_b\, 12$ $\boxed{y_t = 14.1}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1582}$ in^3

$M_{cr\_s} = 1.1\, f_r\, S_c\, \dfrac{1}{12}$ from E14-4.7.1.3 $\boxed{M_{cr\_s} = 65.1}$ kip-ft/ft

$\boxed{1.33\, M_u = 125.0}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = "OK"}$

Check the Service Ib crack control requirements in accordance with **LRFD [5.7.3.4]**

$$\rho = \frac{A_s}{d_s\ b}$$ $\boxed{\rho = 0.00326}$

$$n = \frac{E_s}{E_c}$$ $\boxed{n = 8.09}$

$$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n$$ $\boxed{k = 0.205}$

$$j = 1 - \frac{k}{3}$$ $\boxed{j = 0.932}$

$$d_c = cover + \frac{Bar_D}{2}$$ $\boxed{d_c = 2.6}$ in

$$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12 \le 0.6\ f_y$$ $\boxed{f_{ss} = 31.0}$ ksi $\le 0.6\ f_y$ O.K.

$$h = T_b\ 12$$

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)}$$ $\boxed{\beta_s = 1.1}$

$\gamma_e = 1.00$ for Class 1 exposure

$$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c$$ $\boxed{s_{max} = 14.6}$ in

$\boxed{s = 12.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-4.7.2.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-4.7.3 Temperature and Shrinkage Steel

Evaluate temperature and shrinkage requirements

E14-4.7.3.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required.

E14-4.7.3.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with AASTHO **LRFD [5.10.8]** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided) $\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 258.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right) f_y}$$ Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \le A_s \le 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

Check the maximum spacing requirements

$s_1 = \min\left(3\ h_s, 18\right)$ $\boxed{s_1 = 18.0}$ in

$$s_2 = \begin{vmatrix} 12 & \text{if } h_s > 18 \\ s_1 & \text{otherwise} \end{vmatrix}$$ For walls and footings (in) $\boxed{s_2 = 18.0}$ in

$s_{max} = \min\left(s_1, s_2\right)$ $\boxed{s_{max} = 18.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-4.8 Summary of Results

List summary of results.

E14-4.8.1 Summary of External Stability

Based on the defined project parameters the following external stability checks have been satisfied:

External Check	CDR
	Strength I
Bearing	1.46
Eccentricity	> 10
Sliding	1.12

Table E14-4.8-1
Summary of External Stability Computations

E14-4.8.2 Summary of Wall Strength Design

The required wall reinforcing from the previous computations are presented in Figure E14-6.9-1.

E14-4.8.3 Drainage Design

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are met by providing granular, free draining backfill material with a pipe underdrain located at the bottom of the wall (Assumed wall is adjacent to sidewalk) as shown in Figure E14-4.9-1.

E14-4.9 Final Cast-In-Place Concrete Wall Schematic

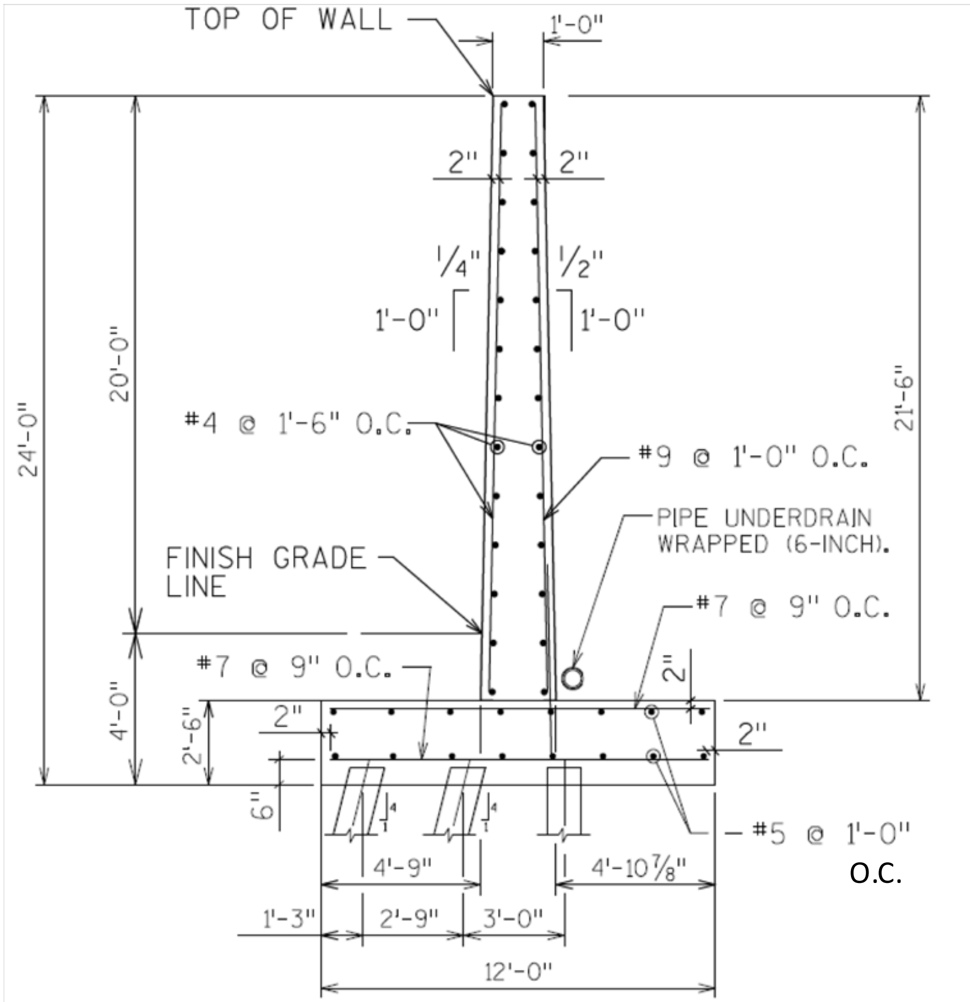

Figure E14-4.9-1
Cast-In-Place Wall Schematic

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E14-5 Sheet Pile Wall, LRFD

General

This example shows design calculations for permanent sheet pile walls conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. *(Example is current through LRFD Fifth Edition - 2010)*

Sample design calculations for required embedment depth and determining preliminary design sections will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.10.5 are used for the wall design.

E14-5.1 Establish Project Requirements

The following example is for a permanent cantilever sheet pile wall penetrating sand and having the low water level at the dredge line as shown in Figure E14-5.1-1. External stability and structural components are the designer's (WisDOT/consultant) responsibility.

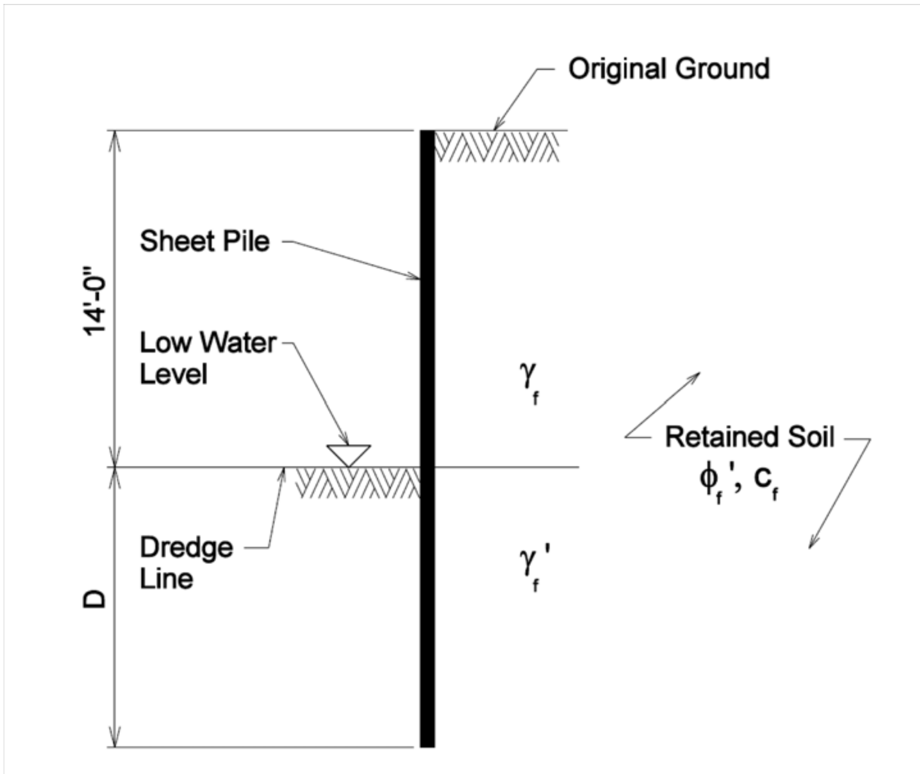

Figure E14-5.1-1
Cantilever Sheet Pile Wall with Horizontal Backslope

Wall Geometry

$H = 14$	Design wall height, ft
$\theta = 90$ deg	Angle of back face of wall to horizontal
$\beta = 0$ deg	Inclination of ground slope behind face of wall (horizontal)

E14-5.2 Design Parameters

Project Parameters

$Design\_Life = 75$	Wall design life (min), years **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Soil Design Parameters

$\phi_f = 35$ deg	Angle of internal friction
$\gamma = 0.115$	Unit weight of soil, kcf
$\gamma_w = 0.0624$	Unit weight of water, kcf
$\gamma' = \gamma - \gamma_w$	Effective unit weight of soil, kcf
$\boxed{\gamma' = 0.053}$	
$c = 0$ psf	Cohesion, psf

Live Load Surcharge Parameters

$SUR = 0.100$	Live load surcharge for walls without traffic, ksf (14.4.5.4.2)

E14-5.3 Establish Earth Pressure Diagram

In accordance with **LRFD [3.11.5.6]** "simplified" and "conventional" methods may be used for lateral earth pressure distributions. This example will use the "simplified" method as shown in **LRFD [Figure 3.11.5.3-2]**. The "conventional" method would result in a more exact solution and is based on Figure E14-5.3-1(b) lateral load distributions.

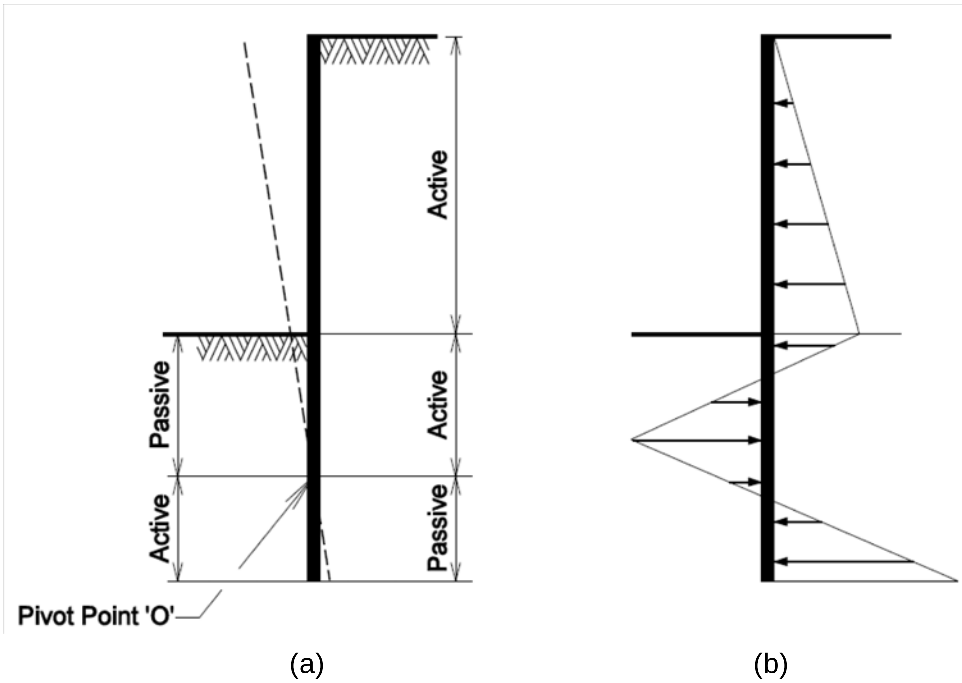

Figure E14-5.3-1
Cantilever Sheet Pile Wall Penetrating a Sand Layer: (a) Wall Yielding Pattern and Earth Pressure Zones; (b) Conventional Net Earth Pressure Distribution (After Das, 2007).

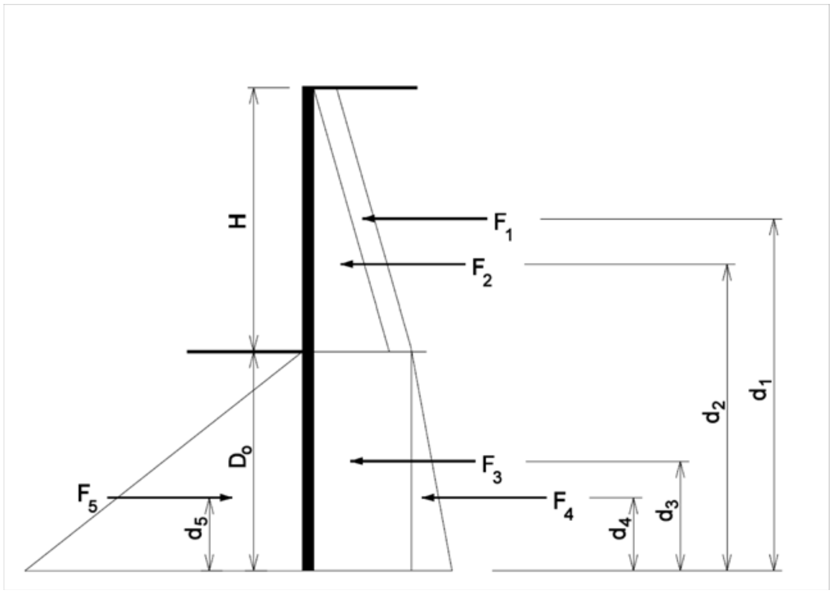

Figure E14-5.3-2
Cantilever Sheet Pile Wall Free-Body Diagram - Simplified Method

E14-5.4 Permanent and Transient Loads

In this example, horizontal earth pressures 'EH' will be used as shown in Figure E14-5.3-1(b). For simplicity, no transient, vertical or surcharge loads are present in this example.

E14-5.4.1 Compute Active Earth Pressure

Compute the coefficient of active earth pressure using Rankine Theory

$$\phi_f = 35 \text{ deg}$$

$$k_a = \tan\left(45 \text{ deg} - \frac{\phi_f}{2}\right)^2 \qquad \boxed{k_a = 0.271}$$

E14-5.4.2 Compute Passive Earth Pressure

Compute the coefficient of passive earth pressure using Rankine Theory

$$\phi_f = 35 \text{ deg}$$

$$k_p = \tan\left(45 \text{ deg} + \frac{\phi_f}{2}\right)^2 \qquad \boxed{k_p = 3.690}$$

E14-5.4.3 Compute Factored Loads

The active earth pressure is factored by its appropriate LRFD load type 'EH' **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. Where as the passive earth pressure is factored by its appropriate resistance factor **LRFD [Table 11.5.7-1]**.

Compute the factored active earth pressure coefficient, K_a

$k_a = 0.271$ Unfactored active earth pressure coefficient

$\gamma_{EH} = 1.50$ Horizontal earth pressure load factor (maximum)

$K_a = \gamma_{EH}\, k_a$ Factored active earth pressure coefficient $\boxed{K_a = 0.406}$

Compute the factored passive earth pressure coefficient, K_p

$k_p = 3.69$ Unfactored passive earth pressure coefficient

$\phi_p = 0.75$ Nongravity cantilevered wall resistance factored for flexural capacity of a vertical element **LRFD [Table 11.5.7-1]**

$K_p = \phi_p\, k_p$ Factored passive earth pressure coefficient $\boxed{K_p = 2.768}$

E14-3.5 Compute Wall Embedment Depth and Factored Bending Moment

Compute the required embedment depth, D_o, corresponding to the depth where the factored active and passive moments are in equilibrium from Figure E14-5.3-2. Trial-and-error is used to determine the depth by adjusting D_o in the following equations:

$D_o = 27.5$ ft

Force (factored)

$F_1 = -(K_a\ SUR)\ H$ $\boxed{F_1 = -0.57}$ kip/ft

$F_2 = \frac{-1}{2}(\gamma\ K_a\ H)\ H$ $\boxed{F_2 = -4.58}$ kip/ft

$F_3 = -(\gamma\ K_a\ H + K_a\ SUR)\ D_o$ $\boxed{F_3 = -19.11}$ kip/ft

$F_4 = \frac{-1}{2}(\gamma'\ K_a\ D_o)\ D_o$ $\boxed{F_4 = -8.08}$ kip/ft

$F_5 = \frac{1}{2}(\gamma'\ K_p\ D_o)\ D_o$ $\boxed{F_5 = 55.05}$ kip/ft

Moment Arm			Moment (factored)		
$d_1 = \frac{H}{2} + D_o$	$\boxed{d_1 = 34.5}$	ft	$M_1 = F_1\ d_1$	$\boxed{M_1 = -19.6}$	kip-ft/ft
$d_2 = \frac{H}{3} + D_o$	$\boxed{d_2 = 32.2}$	ft	$M_2 = F_2\ d_2$	$\boxed{M_2 = -147.4}$	kip-ft/ft
$d_3 = \frac{D_o}{2}$	$\boxed{d_3 = 13.8}$	ft	$M_3 = F_3\ d_3$	$\boxed{M_3 = -262.8}$	kip-ft/ft
$d_4 = \frac{D_o}{3}$	$\boxed{d_4 = 9.2}$	ft	$M_4 = F_4\ d_4$	$\boxed{M_4 = -74.1}$	kip-ft/ft
$d_5 = \frac{D_o}{3}$	$\boxed{d_5 = 9.2}$	ft	$M_5 = F_5\ d_5$	$\boxed{M_5 = 504.6}$	kip-ft/ft

$\Sigma M = M_1 + M_2 + M_3 + M_4 + M_5$ (Approximately equal to zero) $\boxed{\Sigma M = 0.66}$ kip-ft/ft

Capacity:Demand Ratio (CDR) at D_o

$M_a = M_1 + M_2 + M_3 + M_4$ Factored active moments $\boxed{M_a = -503.9}$ kip-ft/ft

$M_p = M_5$ Factored passive moments $\boxed{M_p = 504.6}$ kip-ft/ft

$CDR = \left|\frac{M_p}{M_a}\right|$ $\boxed{CDR = 1.00}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

Compute the required embedment depth, D. Since the wall embedment depth uses the Simplified Method with continuous vertical elements a 20% increase in embedment will be included as shown in **LRFD [Figure 3.11.5.6-3]**.

$D = 1.2\ D_o$ $\boxed{D = 33.00}$ ft

Compute the location of the maximum bending moment, M_{max}, corresponding to the depth where the factored active and passive lateral forces are in equilibrium from Figure E14-5.3-2. Trial-and-error is used to determine the depth by adjusting D_o in the following equations:

$D_o = 16.3$ ft

Force (factored)

$F_1 = -(K_a\ SUR)\ H$ $\boxed{F_1 = -0.57}$ kip/ft

$F_2 = \frac{-1}{2}(\gamma\ K_a\ H)\ H$ $\boxed{F_2 = -4.58}$ kip/ft

$F_3 = -(\gamma\ K_a\ H + K_a\ SUR)\ D_o$ $\boxed{F_3 = -11.33}$ kip/ft

$F_4 = \frac{-1}{2}(\gamma'\ K_a\ D_o)\ D_o$ $\boxed{F_4 = -2.84}$ kip/ft

$F_5 = \frac{1}{2}(\gamma'\ K_p\ D_o)\ D_o$ $\boxed{F_5 = 19.34}$ kip/ft

$\Sigma F = F_1 + F_2 + F_3 + F_4 + F_5$ (Approximately equal to zero) $\boxed{\Sigma F = 0.02}$ kip-ft/ft

Moment Arm		Moment (factored)	
$d_1 = \frac{H}{2} + D_o$	$\boxed{d_1 = 23.3}$ ft	$M_1 = F_1\ d_1$	$\boxed{M_1 = -13.3}$ kip-ft/ft
$d_2 = \frac{H}{3} + D_o$	$\boxed{d_2 = 21.0}$ ft	$M_2 = F_2\ d_2$	$\boxed{M_2 = -96.1}$ kip-ft/ft
$d_3 = \frac{D_o}{2}$	$\boxed{d_3 = 8.2}$ ft	$M_3 = F_3\ d_3$	$\boxed{M_3 = -92.3}$ kip-ft/ft
$d_4 = \frac{D_o}{3}$	$\boxed{d_4 = 5.4}$ ft	$M_4 = F_4\ d_4$	$\boxed{M_4 = -15.4}$ kip-ft/ft
$d_5 = \frac{D_o}{3}$	$\boxed{d_5 = 5.4}$ ft	$M_5 = F_5\ d_5$	$\boxed{M_5 = 105.1}$ kip-ft/ft

$\Sigma M = M_1 + M_2 + M_3 + M_4 + M_5$ $\boxed{\Sigma M = -112.0}$ kip-ft/ft

$M_{max} = |\Sigma M|$ $\boxed{M_{max} = 112.0}$ kip-ft/ft

Figure E14-5.5-1 tabulates the above computations in a spreadsheet for varying embedment depths.

D_o	F_1	F_2	F_3	F_4	F_5	d_1	d_2	d_3	d_4	d_5	Fa	Fp	Fa+Fp	M_1	M_2	M_3	M_4	M_5	M_a	M_p	CDR	M_a+M_p
0	-0.6	-4.6	0.0	0.0	0.0	7.0	4.7	0.0	0.0	0.0	-5.2	0.0	-5.2	-4	-21	0	0	0	-25	0	0.0	-25.4
2	-0.6	-4.6	-1.4	0.0	0.3	9.0	6.7	1.0	0.7	0.7	-6.6	0.3	-6.3	-5	-31	-1	0	0	-37	0	0.0	-36.9
4	-0.6	-4.6	-2.8	-0.2	1.2	11.0	8.7	2.0	1.3	1.3	-8.1	1.2	-6.9	-6	-40	-6	0	2	-52	2	0.0	-50.2
6	-0.6	-4.6	-4.2	-0.4	2.6	13.0	10.7	3.0	2.0	2.0	-9.7	2.6	-7.1	-7	-49	-13	-1	5	-70	5	0.1	-64.3
8	-0.6	-4.6	-5.6	-0.7	4.7	15.0	12.7	4.0	2.7	2.7	-11.4	4.7	-6.7	-9	-58	-22	-2	12	-91	12	0.1	-78.2
10	-0.6	-4.6	-7.0	-1.1	7.3	17.0	14.7	5.0	3.3	3.3	-13.2	7.3	-5.9	-10	-67	-35	-4	24	-115	24	0.2	-90.9
12	-0.6	-4.6	-8.3	-1.5	10.5	19.0	16.7	6.0	4.0	4.0	-15.0	10.5	-4.5	-11	-76	-50	-6	42	-143	42	0.3	-101.4
14	-0.6	-4.6	-9.7	-2.1	14.3	21.0	18.7	7.0	4.7	4.7	-17.0	14.3	-2.7	-12	-86	-68	-10	67	-175	67	0.4	-108.8
16.3	**-0.6**	**-4.6**	**-11.3**	**-2.8**	**19.3**	**23.3**	**21.0**	**8.2**	**5.4**	**5.4**	**-19.3**	**19.3**	**0.0**	**-13**	**-96**	**-92**	**-15**	**105**	**-217**	**105**	**0.5**	**-112.0**
18	-0.6	-4.6	-12.5	-3.5	23.6	25.0	22.7	9.0	6.0	6.0	-21.1	23.6	2.5	-14	-104	-113	-21	142	-251	142	0.6	-110.0
20	-0.6	-4.6	-13.9	-4.3	29.1	27.0	24.7	10.0	6.7	6.7	-23.3	29.1	5.8	-15	-113	-139	-29	194	-296	194	0.7	-101.8
22	-0.6	-4.6	-15.3	-5.2	35.2	29.0	26.7	11.0	7.3	7.3	-25.6	35.2	9.6	-17	-122	-168	-38	258	-345	258	0.7	-86.5
24	-0.6	-4.6	-16.7	-6.2	41.9	31.0	28.7	12.0	8.0	8.0	-28.0	41.9	13.9	-18	-131	-200	-49	335	-398	335	0.8	-63.0
26	-0.6	-4.6	-18.1	-7.2	49.2	33.0	30.7	13.0	8.7	8.7	-30.4	49.2	18.8	-19	-140	-235	-63	426	-457	426	0.9	-30.4
27.5	**-0.6**	**-4.6**	**-19.1**	**-8.1**	**54.9**	**34.5**	**32.1**	**13.7**	**9.2**	**9.2**	**-32.3**	**54.9**	**22.6**	**-20**	**-147**	**-262**	**-74**	**503**	**-503**	**503**	**1.0**	**0.0**
30	-0.6	-4.6	-20.9	-9.6	65.5	37.0	34.7	15.0	10.0	10.0	-35.6	65.5	29.9	-21	-159	-313	-96	655	-589	655	1.1	66.2
32	-0.6	-4.6	-22.2	-10.9	74.5	39.0	36.7	16.0	10.7	10.7	-38.3	74.5	36.2	-22	-168	-356	-117	795	-663	795	1.2	132.2

Results Tabulated Above Values		
Required Embedment Depth, D_o (M_p/M_a>1)=	**27.47**	ft
Actual Embedment (1.2*D_o) =	**32.96**	ft
Maximum Factored Moment Location (F_a+F_p=0) =	**16.30**	ft
Maximum Factored Design Moment=	**112.0**	kip-ft/ft

Figure E14-5.5-1
Design Analysis for Cantilever Sheet Pile Wall

E14-5.6 Compute the Required Flexural Resistance

The following is a design check for flexural resistance:

$$M_{max} \le \phi_f M_n \qquad \phi_f M_n = \phi_f F_y Z$$

$M_{max} = 112.0$ kip-ft/ft

$\phi_f = 0.90$ Resistance factor for flexure (based on nongravity cantilevered walls for the flexural capacity of vertical elements **LRFD [Table 11.5.7-1]**)

M_n Nominal flexural resistance of the section

$F_y = 50$ Steel yield stress, ksi (assumed A572 Grade 50)

Z Plastic section modulus (in^3/ft)

$$Z_{reqd} = \frac{M_{max}\ 12}{\phi_f\ F_y} \qquad \boxed{Z_{reqd} = 29.87}\ \text{in}^3/\text{ft}$$

Based on this minimum section modulus a preliminary sheet pile section PZ-27 (Z=36.49 in^3/ft) is selected. Additional design checks shall be made based on project requirements.

E14-5.7 Final Sheet Pile Wall Schematic

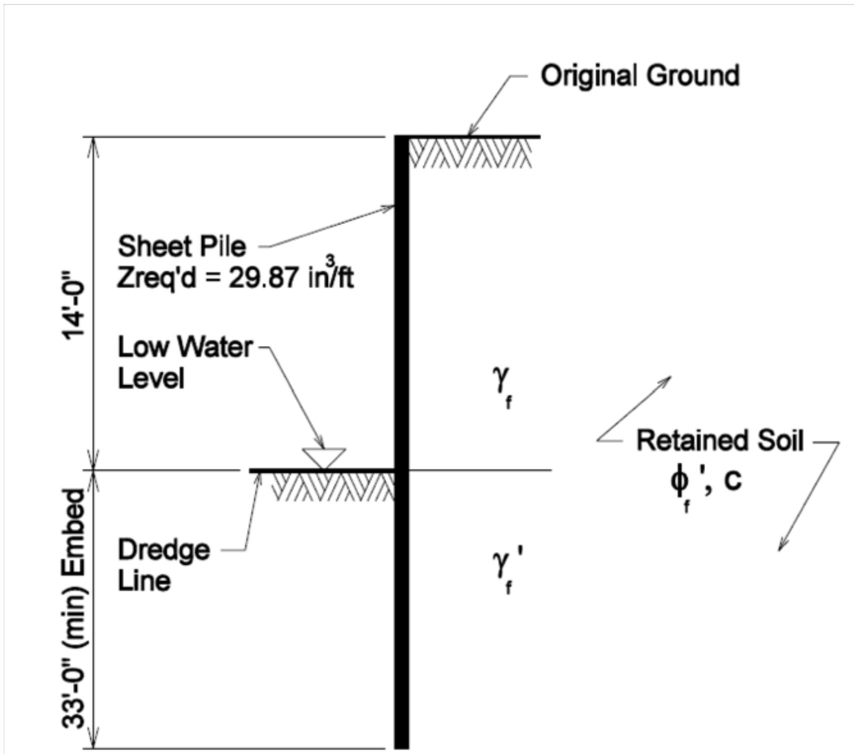

Figure E14-5.7-1
Cantilever Sheet Pile Wall Schematic

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15.1 Grade Separations

In general, there are three types of slope paving used at the abutments of grade separation bridges; cast-in-place concrete, crushed aggregate or select crushed material, and concrete block. Concrete cast-in-place is used in urban areas or where appearance is a prime consideration. Bituminous stabilized crushed aggregate or select crushed material is used in rural areas or where appearance is not as important. Refer to Slope Paving Structures standard details for additional information.

Precast concrete blocks (approximately 4 x 16 x 24 inches) were the standard applications during the late 50's and early 60's. Many blocks settled or washed out of place due to erosion of bedding under the blocks. They are no longer specified except on widening jobs to match existing slope paving.

15.2 Stream Crossing

Heavy riprap is used for slope protection at stream crossings due to its superior performance over medium random riprap. In general, due to the favorable performance and relatively low cost of geotextile fabrics, they are used under heavy riprap whenever heavy riprap is specified for a project.

Many factors influence the criteria used to select end slopes. These include:

1. The type of soil. (granular, cohesive, borrow or in-situ)
2. Type and impact of a failure to stream/roadway/structure.
3. Type of abutment foundation support. (spread footings vs. piles)
4. History of the existing slopes at structure replacement sites.
5. Additional bridge costs when structures are lengthened due to flatter slopes.

The current standard for slopes is 1.5:1. However, for conditions where the vertical height of fill from berm to toe of slope exceeds 15 feet, consider flattening slopes to 2:1, or breaking up the slope by providing a plateau area halfway through the slope.

Furthermore, if slope soil materials are “fairly granular”, use current standards. For other soil types, flatten slopes to 2:1. If existing problems are noted or there is no historical information at the site, analyze site geometry to determine slope.

Refer to the Standard for Placement of Heavy Riprap at River Crossings for placement of heavy riprap. Any additional riprap not covered by the standard is not part of the structure plans.

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17.1 Design Method

17.1.1 Design Requirements

All new structures and deck replacements are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*. Bridge rehabilitations and widenings are to be designed using either LFD or LRFD, at the designer's option.

LRFD utilizes load combinations called limit states which represent the various loading conditions which structural materials must be able to withstand. Limit states have been established in four major categories – strength, service, fatigue and extreme event. Different load combinations are used to analyze a structure for certain responses such as deflections, permanent deformations, ultimate strength and inelastic responses without failure. When all applicable limit states and combinations are satisfied, a structure is deemed acceptable under the LRFD design philosophy.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

17.1.2 Rating Requirements

Rating factors, RF, for inventory and operating rating are shown on the plans. Ratings will be based on *The Manual for Bridge Evaluation*, hereafter referred to as *AASHTO MBE*. See Chapter 45 – Bridge Rating for rating requirements. Existing ratings for rehabilitation projects where the final ratings will not change should be taken from HSI and placed on the final plans. See Section 6.2.2.3.4 for more information.

17.1.2.1 Standard Permit Design Check

New structures are also to be checked for the Wisconsin Standard Permit Vehicle (Wis-SPV). The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface. This truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the bridge, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM.

The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

See Chapter 45 – Bridge Rating for details about the Wisconsin Standard Permit Vehicle and calculating the maximum load for this permit vehicle.

17.2 LRFD Requirements

17.2.1 General

For superstructure member design, the component dimensions and the size and spacing of reinforcement shall be selected to satisfy the following equation for all appropriate limit states, as presented in **LRFD [1.3.2.1]**:

$$Q = \sum \eta_i \, \gamma_i \, Q_i \le \phi \, R_n = R_r$$

Where:

η_i	=	Load modifier (a function of η_D, η_R, and η_I)
γ_i	=	Load factor
Q_i	=	Force effect: moment, shear, stress range or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance: resistance of a component to force effects
R_r	=	Factored resistance = ϕR_n

17.2.2 WisDOT Policy Items

WisDOT policy items:

Set the value of the load modifier, η_i (see **LRFD [1.3.2.1]**), and its factors, η_D, η_R and η_I, all equal to 1.00.

Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects.

17.2.3 Limit States

The following limit states (as defined in **LRFD [3.4.1]**) are utilized by WisDOT in the design of bridge superstructures.

17.2.3.1 Strength Limit State

The strength limit state shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a bridge is expected to experience during its design life. The total factored force effect must not exceed the factored resistance.

Strength I is used for the ultimate capacity of structural members and relates to the normal vehicular use of the bridge without wind.

Strength II is not typically used by WisDOT. However, Wisconsin Standard Permit Vehicle (Wis-SPV) must be checked in accordance with Chapter 45 – Bridge Rating.

Strength III is not typically used as a final-condition design check by WisDOT.

WisDOT policy item:

Strength III can be used as a construction check for steel girder bridges with wind load but no live load.

Strength IV is not typically used by WisDOT. Spans > 300 ft. should include this limit state.

Strength V relates to the normal vehicular use of the bridge with wind speed (3-second gust) as specified in **LRFD [3.8]**. This limit state is used in the design of steel structures to check lateral bending stresses in the flanges.

17.2.3.2 Service Limit State

The service limit state shall be applied to restrict stress, deformation and crack width under regular service conditions. The total factored force effect must not exceed the factored resistance.

Service I relates to the normal vehicular use of the bridge. This limit state is used to check general serviceability requirements such as deflections and crack control. This load combination is also used to check compressive stresses in prestressed concrete components.

Service II is intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live loads.

Service III is used to check the tensile stresses in prestressed concrete superstructures with the objective of crack control.

17.2.3.3 Fatigue Limit State

The fatigue limit state shall be applied as a restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles. The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge. The total factored force effect must not exceed the factored resistance.

Fatigue I is related to infinite load-induced fatigue life. This load combination should be checked for longitudinal slab bridge reinforcement and longitudinal continuity reinforcement on prestressed concrete girder and steel girder bridges. Fatigue I is used for steel girder structures to determine whether or not a tensile stress could exist at a particular location. This load combination is also used for any fracture-critical members as well as components and details not meeting the requirements for Fatigue II.

Fatigue II is related to finite load-induced fatigue life. If the projected 75-year single lane Average Daily Truck Traffic is less than or equal to a prescribed value for a given component or detail, that component or detail should be designed for finite life using the Fatigue II load combination.

17.2.3.4 Extreme Event Limit State

The extreme event limit state shall be applied for deck overhang design as specified in Table 17.6-1. For the extreme limit state, the applied loads for deck overhang design are horizontal and vertical vehicular collision forces. These forces are checked at the inside face of the barrier, the design section for the overhang and the design section for the first bay, as described in 17.6.

Extreme Event II is used to design deck reinforcement due to vehicular collision forces.

17.2.4 Design Loads

In LRFD design, structural materials must be able to resist their applied design loads. Two general types of design loads are permanent and transient. Permanent loads include dead load and earth load. Transient loads include live loads, wind, temperature, braking force and centrifugal force.

17.2.4.1 Dead Loads

Superstructures must be designed to resist dead load effects. In LRFD, dead load components consist of DC and DW dead loads. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. Different load factors are used for DC and DW dead loads, as described in 17.2.5, to account for the differences in the predictability of the loading. In addition, some dead loads are resisted by the non-composite section and other dead loads are resisted by the composite section.

Table 17.2-1 summarizes the various dead load components that are commonly included in beam-on-slab superstructure design. For slab structures, all loads presented in this table are resisted by the slab.

Dead Load Resisted By	Type of Load Factor	
	DC	DW
Non-composite section	• Girder • Concrete deck • Concrete haunch • Miscellaneous dead load (including diaphragms, cross-frames, stiffeners, etc.)	
Composite section	• Concrete parapets • Sidewalks • Medians	• Future wearing surface • Utilities

Table 17.2-1
Dead Load Components

In the absence of more precise information, **LRFD [Table 3.5.1-1]** provides some guidance for typical unit weights.

Dead loads should be computed based on the following:

The uniform dead load of the deck or slab is determined using the concrete unit weight and simple beam distribution. A concrete unit weight of 0.150 kcf should be used.

The weight of the concrete haunch is determined by estimating the minimum haunch depth at 2" at the edge of girder and the width equal to the largest top flange of the supporting member. The cross slope, girder camber and profile grade line must be considered.

The weights of steel beams and girders are determined from the AISC Manual of Steel Construction. Haunched webs of plate girders are converted to an equivalent uniform partial dead load.

The weight of secondary steel members such as bracing, shear studs and stiffeners can be estimated at 30 plf for interior girders and 20 plf for exterior girders.

The weight of prestressed concrete girders is presented in the Standard Details.

A dead load of 20 psf is added to account for a future wearing surface. Future wearing surface is applied from face to face of curb and shall not be applied to sidewalks.

The weight of the parapets, sidewalks, barriers and medians shall be based on a unit weight of 0.150 kcf. The weight per foot for the standard parapets are presented in the Standard Details.

17.2.4.2 Traffic Live Loads

The design vehicular load currently used by AASHTO is designated as HL-93, in which "HL" is an abbreviation for highway loading and "93" represents the year of 1993 in which the loading was accepted by AASHTO. The HL-93 live load consists of the following load types:

- Design truck
- Design tandem
- Design lane
- Double truck
- Fatigue truck

Using these basic load types, *AASHTO LRFD* combines and scales them to create live load combinations that apply to different limit states, as described in **LRFD [3.6.1]** and as shown below.

17.2.4.2.1 Design Truck

The design truck has three axles, with axle loads and spacings as presented in Figure 17.2-1.

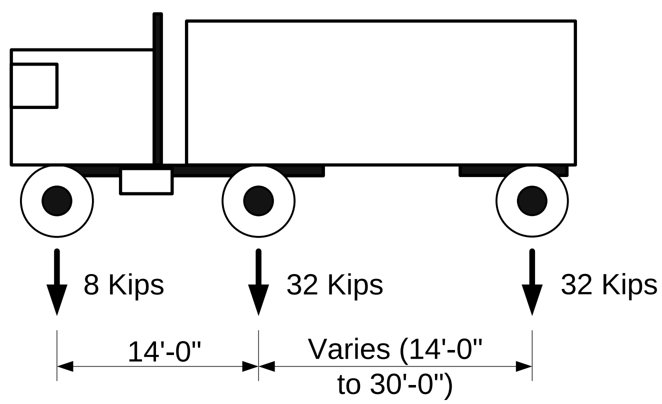


Figure 17.2-1
Design Truck

The axle spacing between the second and third axles is selected such that the maximum effect is achieved. The minimum axle spacing of 14 feet usually controls. However, a situation in which an axle spacing greater than 14 feet may control is for a continuous short-span bridge in which the maximum negative moment at the pier is being computed and the second and third axles are positioned in different spans. The design truck is described in **LRFD [3.6.1.2.2]**.

17.2.4.2.2 Design Tandem

The design tandem has two axles, each with a loading of 25 kips and an axle spacing of 4 feet, as presented in Figure 17.2-2. The design tandem is described in **LRFD [3.6.1.2.3]**.

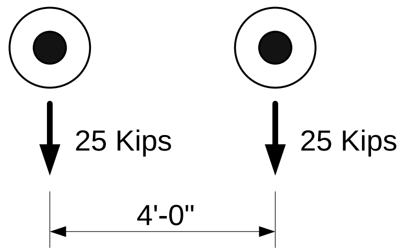


Figure 17.2-2
Design Tandem

WisDOT policy item:

WisDOT does not consider the use of dual tandems for negative moments and reactions, as suggested in **LRFD [C3.6.1.3.1]**. The design engineer shall receive direction from the owner and the BOS if this load is to be applied.

17.2.4.2.3 Design Lane

The design lane has a uniform load of 0.64 kips per linear foot, distributed in the longitudinal direction, as presented in Figure 17.2-3. The design lane is described in **LRFD [3.6.1.2.4]**.

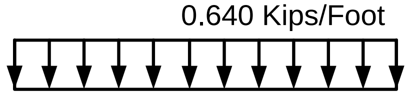


Figure 17.2-3
Design Lane

17.2.4.2.4 Double Truck

For negative moments and reactions at piers, a third condition is also considered. Two design trucks are applied, with a minimum headway between the front and rear axles of the two trucks equal to 50 feet. The rear axle spacing of the two trucks is set at a constant 14 feet. 90% of the effect of the two design trucks is combined with 90% of the design lane load, as presented in Figure 17.2-4. This loading is described in **LRFD [3.6.1.3.1]**.

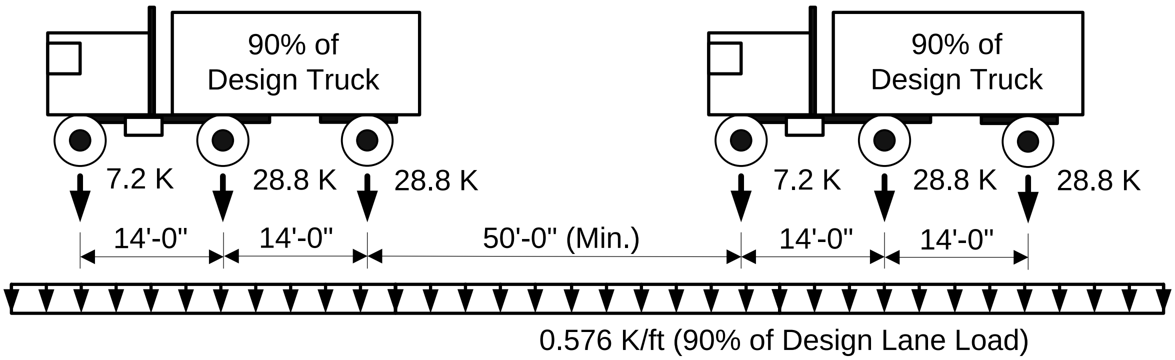


Figure 17.2-4
Double Truck

17.2.4.2.5 Fatigue Truck

The fatigue truck consists of one design truck similar to that described in 17.2.4.2.1 but with a constant spacing of 30 feet between the 32-kip axles, as presented in Figure 17.2-5. The fatigue truck is described in **LRFD [3.6.1.4.1]**.

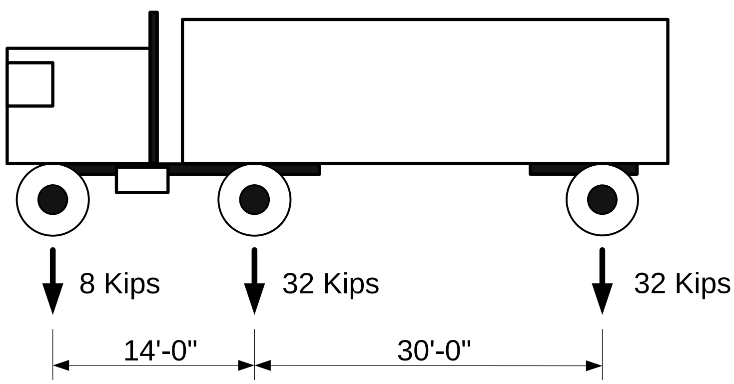


Figure 17.2-5
Fatigue Truck

17.2.4.2.6 Live Load Combinations

The live load combinations used for design are presented in Table 17.2-2.

Live Load Combination	Description	Reference
LL#1	Design tandem (+ IM) + design lane load	**LRFD [3.6.1.3.1]**
LL#2	Design truck (+ IM) + design lane load	**LRFD [3.6.1.3.1]**

LL#3	Double truck [90% of two design trucks (+ IM) + 90% of design lane load] *	**LRFD [3.6.1.3.1]**
LL#4	Fatigue truck (+ IM)	**LRFD [3.6.1.4.1]**
LL#5	Design truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6	25% [design truck (+ IM)] + design lane load	**LRFD [3.6.1.3.2]**

* LL#3 is used to calculate negative live load moments between points of contraflexure, as well as reactions at interior supports.

Table 17.2-2
Live Load Combinations

The live load combinations are applied to the limit states as follows:

Strength I – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Strength V – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Service I – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3. However, for live load deflection criteria, the force effects shall be taken as the larger of LL#5 and LL#6.

Service II – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Service III – The live load force effect, Qi, shall be taken as the larger of LL#1, LL#2 and LL#3.

Fatigue (I or II) – The live load force effect, Qi, shall be from a single fatigue truck, LL#4.

Extreme Event II – The live load force effect, Qi, shall be taken as the larger of LL#1 and LL#2.

17.2.4.3 Multiple Presence Factor

The extreme force effect shall be determined by considering each possible combination of the number of loaded lanes multiplied by a corresponding multiple presence factor. This factor accounts for the probability of simultaneous lane occupation by the full HL93 design live load. Note that the multiple presence factor has been included in the approximate equations for distribution factors in **LRFD [4.6.2.2]** and **[4.6.2.3]**, and in 17.2.8 of this manual.

As described in **LRFD [3.6.1.1.2]**, the multiple presence factors, m, have the values as presented in Table 17.2-3

Number of Loaded Lanes	Multiple Presence Factors “m”
1	1.20
2	1.00
3	0.85
>3	0.65

Table 17.2-3
Multiple Presence Factors

17.2.4.4 Dynamic Load Allowance

The HL-93 loading is based on a static live load applied to the bridge. However, in reality, the live load is not static but is moving across the bridge. Since the roadway surface on a bridge is usually not perfectly smooth and the suspension systems of most trucks react to roadway roughness with oscillations, a dynamic load is applied to the bridge and must also be considered with the live load. This is referred to as dynamic load allowance.

As described in **LRFD [3.6.2]**, the dynamic load allowance has values as presented in Table 17.2-4.

Component	Limit State	Dynamic Load Allowance, IM
Deck joints	All limit states	75%
All other components	Fatigue and fracture limit states	15%
	All other limit states	33%

Table 17.2-4
Dynamic Load Allowance

Applying these specifications to the live load combinations listed in Table 17.2-2:

IM = 15% for fatigue truck (LL#4)

IM = 33% for all other live load combinations (LL#1, LL#2, LL#3, LL#5 and LL#6)

Where IM is required, multiply the loads by (1 + IM/100) to include the dynamic effects of the load.

It is important to note that the dynamic load allowance is applied only to the design truck and design tandem. The dynamic load allowance is not applied to the design lane load or to pedestrian loads.

17.2.4.5 Pedestrian Loads

For bridges designed for both vehicular and pedestrian load, a pedestrian load of 75 psf is used, as specified in **LRFD [3.6.1.6]**. However, for bridges designed exclusively for pedestrian and/or bicycle traffic, a live load of 90 psf is used. Consideration should also be given to maintenance vehicle loads as specified in Chapter 37 – Pedestrian Bridges.

17.2.5 Load Factors

The load factor, γ_i, is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis and the probability of simultaneous occurrence of different loads.

For the design limit states, the values of γ_i for different types of loads are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. Load factors most commonly used for superstructure design are also presented in Table 17.2-5.

Load Combination	Load Factor, γi				
	DC		DW		LL+IM
	Maximum	Minimum	Maximum	Minimum	
Strength I	1.25	0.90	1.50	0.65	1.75
Strength III	1.25	0.90	1.50	0.65	0.00
Strength V	1.25	0.90	1.50	0.65	1.35
Service I	1.00	1.00	1.00	1.00	1.00
Service II	1.00	1.00	1.00	1.00	1.30
Service III	1.00	1.00	1.00	1.00	0.80
Fatigue I	0.00	0.00	0.00	0.00	1.50
Extreme Event II	1.25	0.90	1.50	0.65	0.50

Table 17.2-5
Load Factors

The maximum and minimum values should be used to maximize the intended effect of the load. An example of the use of minimum load factors is the load factor for dead load when uplift is being checked.

17.2.6 Resistance Factors

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

Resistance factors are presented in **LRFD [1.3.2.1]**, **LRFD [5.5.4.2]**, **LRFD [6.5.4.2]**, **LRFD [6.5.5]** and **LRFD [6.10.1.7]**. The most commonly used resistance factors for superstructure design are also presented in Table 17.2-6.

Limit State	Material	Application	Resistance Factor, ϕ
Strength	Concrete	Flexure (reinforced concrete)	0.90
		Flexure (prestressed concrete)	1.00
		Shear (normal weight)	0.90
		Shear (lightweight)	0.70
	Steel	Flexure	1.00
		Shear	1.00
		Axial compression, steel only	0.90
		Axial compression, composite	0.90
		Tension, fracture in net section	0.80
		Tension, yielding in gross section	0.95
		Bolts bearing on material	0.80
		Shear connectors	0.85
		A325 and A490 bolts in tension	0.80
		A325 and A490 bolts in shear	0.80
		A307 bolts in tension	0.80
		A307 bolts in shear	0.65
		Block shear	0.80
		Web crippling	0.80
		Welds	See **LRFD [6.5.4.2]**
Service	All	All	1.0
Fatigue	All	All	1.0
Extreme Event	All	All	1.0

Table 17.2-6
Resistance Factors

17.2.7 Distribution of Loads for Slab Structures

For slab structures, the distribution of loads is based on strip widths, as illustrated in Figure 17.2-6 through Figure 17.2-11. Figure 17.2-6 and Figure 17.2-7 illustrate the distribution of loads for slab structures with no sidewalks. Figure 17.2-8 and Figure 17.2-9 illustrate the distribution of loads for slab structures with sidewalks. Figure 17.2-10 and Figure 17.2-11 illustrate the distribution of loads for slab structures with raised sidewalks. It should be noted that, although medians are not shown in these figures, medians are treated similar to other superimposed dead loads.

The first step in determining the distribution of loads for slab structures is to compute the strip width, as specified in **LRFD [4.6.2.3]** and **LRFD [4.6.2.1.4]**. Equations for strip widths are also presented in Chapter 18 – Concrete Slab Structure.

For each of the following figures, the distribution of loads for that slab configuration and strip location is described and a general equation is presented immediately below the corresponding figure. In the general equations, it is assumed that dynamic load allowance is applied to the appropriate live load components.

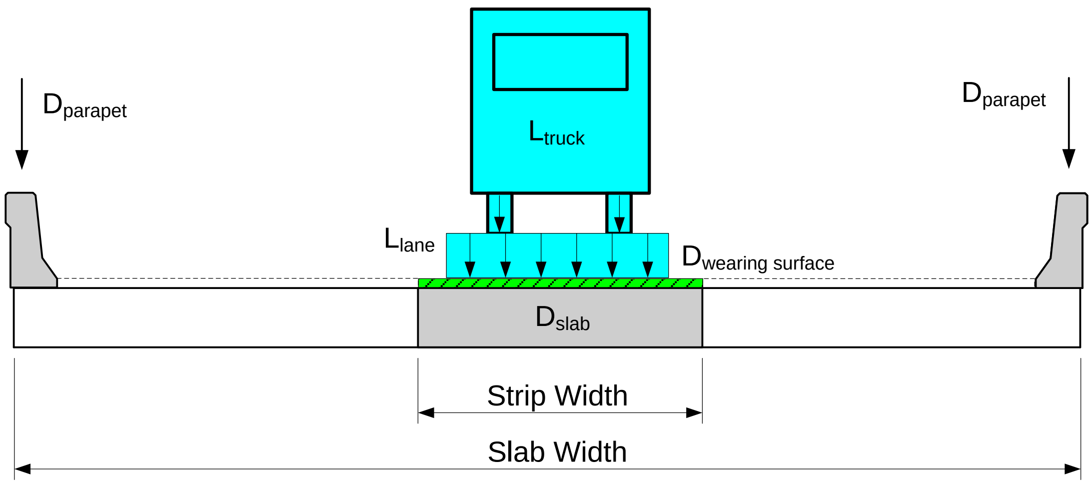


Figure 17.2-6
Distribution of Loads to Interior Strip Width for Slab Structure

The distribution of loads to the interior strip is calculated as follows:

For dead loads, the strip width must resist its self-weight, the future wearing surface and all superimposed dead loads. The superimposed dead loads (including parapets and medians) are distributed uniformly across the entire slab width. The distribution of superimposed dead load to the interior strip width is then computed based on the ratio of the interior strip width to the slab width.

For live loads, one lane of live loading is applied to the interior strip width.

The general equation for loads applied to the interior strip width is as follows:

$$\text{Total Load} = D_{slab} + D_{wearing\ surface} + \left[\left(2\,D_{parapet} \right) \left(\frac{\text{Strip Width}}{\text{Slab Width}} \right) \right] + \left(L_{truck} + L_{lane} \right)$$

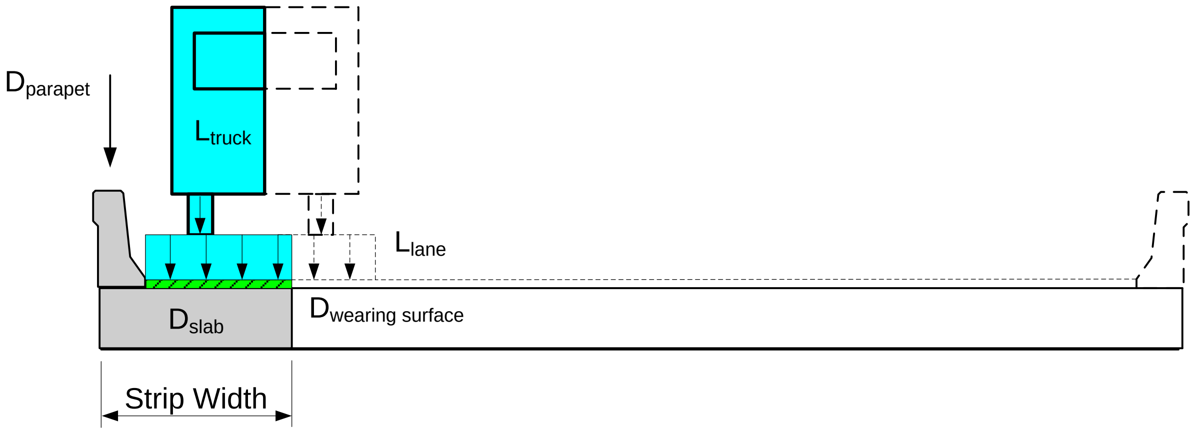


Figure 17.2-7

Distribution of Loads to Exterior Strip Width for Slab Structure

The distribution of loads to the exterior strip is calculated as follows:

For dead loads, the strip width must resist its self-weight and all superimposed dead loads located directly over the strip.

For the design lane load, only the portion of the lane load located directly over the exterior strip width is applied to the exterior strip. For the design vehicle, only half of the axle weights (one line of wheels) are applied to the exterior strip.

The general equation for loads applied to the exterior strip width is as follows:

$$\text{Total Load} = D_{\text{slab}} + \left(D_{\text{wearing surface}} + D_{\text{parapet}}\right)_{\text{directly over strip}} + \left(L_{\text{truck}} + L_{\text{lane}}\right)_{\text{directly over strip}}$$

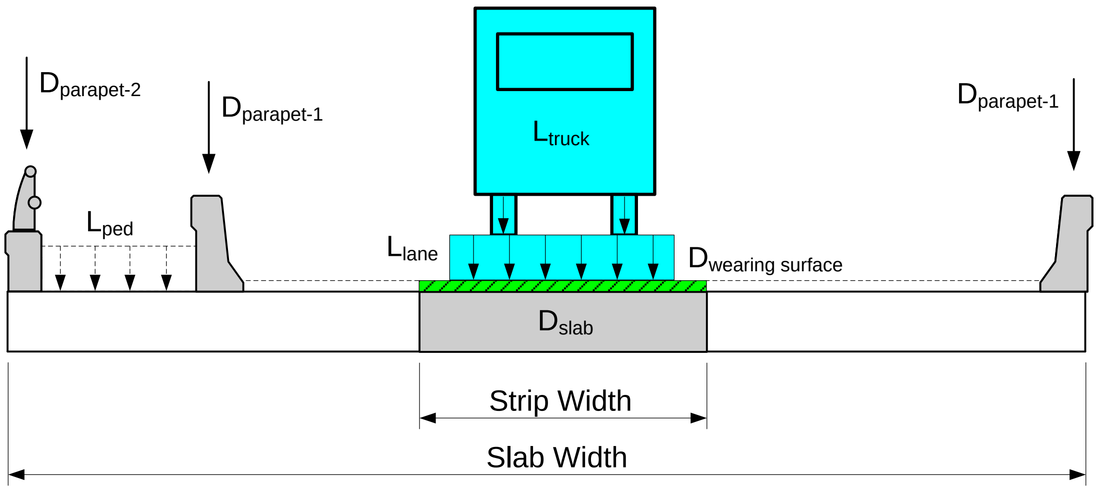


Figure 17.2-8
Distribution of Loads to Interior Strip Width for Slab Structure with Sidewalk

The distribution of loads to the interior strip is calculated as follows:

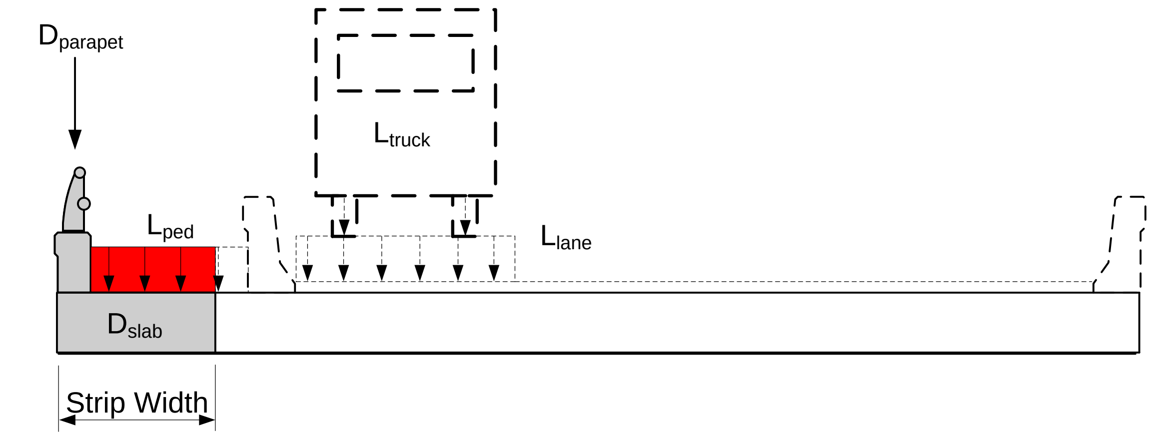
For dead loads, the strip width must resist its self-weight, future wearing surface, and all superimposed dead loads. The superimposed dead loads (including parapets and medians) are distributed uniformly across the entire slab width. The distribution of superimposed dead load to the interior strip width is then computed based on the ratio of the interior strip width to the slab width. Wearing surface is not applied to sidewalks.

For live loads, one lane of live loading is applied to the interior strip width.

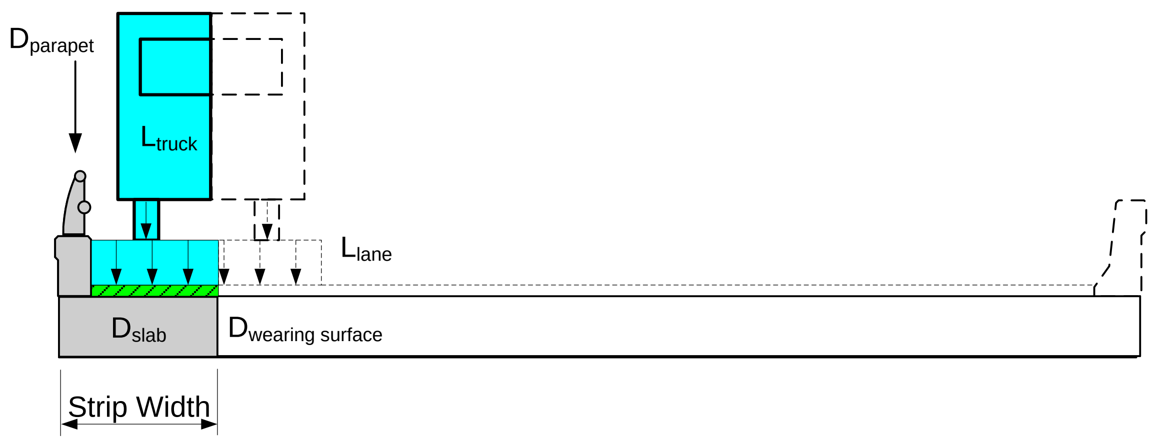
Pedestrian loads are not applied to the interior strip width.

The general equation for loads applied to the interior strip width is as follows:

$$\text{Total Load} = D_{\text{slab}} + D_{\text{wearing surface}} + \left[\left(2\,D_{\text{parapet-1}} + D_{\text{parapet-2}}\right)\left(\frac{\text{Strip Width}}{\text{Slab Width}}\right)\right] + \left(L_{\text{truck}} + L_{\text{lane}}\right)$$



Actual Configuration



Design Configuration

Figure 17.2-9
Distribution of Loads to Exterior Strip Width for Slab Structure with Sidewalk

The distribution of loads to the exterior strip is calculated as follows:

For dead loads, the strip width must resist its self-weight and all superimposed dead loads located directly over the strip. However, it is assumed that the interior parapet is not present.

For live loads, it is assumed that the interior parapet is not present. Therefore, the vehicle and lane are positioned as shown in the Design Configuration portion of the previous figure. For the design lane load, only the portion of the lane load located directly over the exterior strip width is applied to the exterior strip. For the design vehicle, only half of the axle weights (one line of wheels) are applied to the exterior strip.

For pedestrian loads, it is assumed that none are present due to the assumed absence of the interior parapet and the assumed presence of vehicular live load immediately adjacent to the exterior parapet.

The general equation for loads applied to the exterior strip width is as follows:

$$\text{Total Load} = D_{\text{slab}} + \left(D_{\text{wearing surface}} + D_{\text{parapet}}\right)_{\text{directly over strip}} + \left(L_{\text{truck}} + L_{\text{lane}}\right)_{\text{directly over strip}}$$

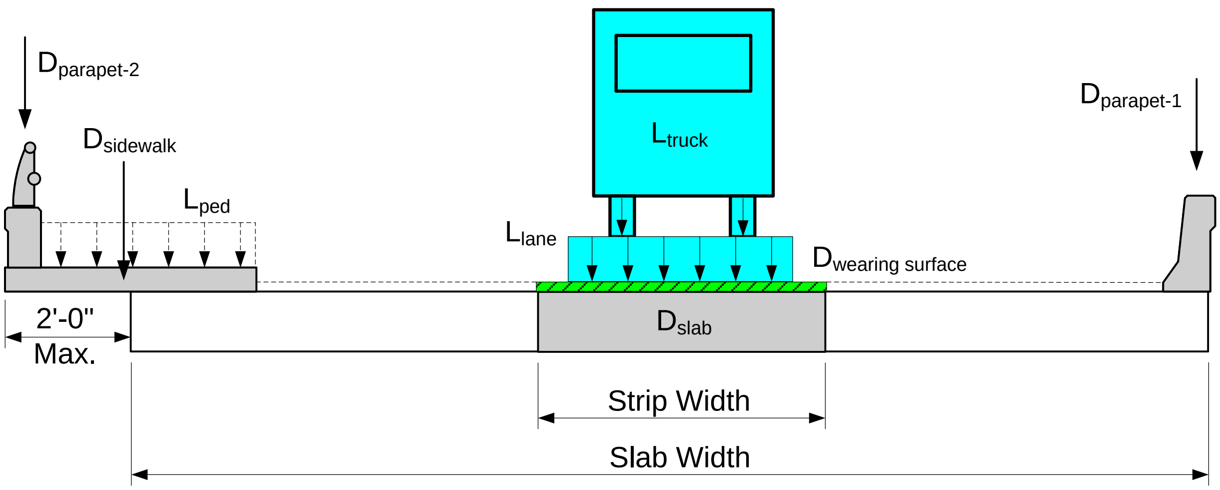


Figure 17.2-10

Distribution of Loads to Interior Strip Width for Slab Structure with Raised Sidewalk

The distribution of loads to the interior strip is calculated as follows:

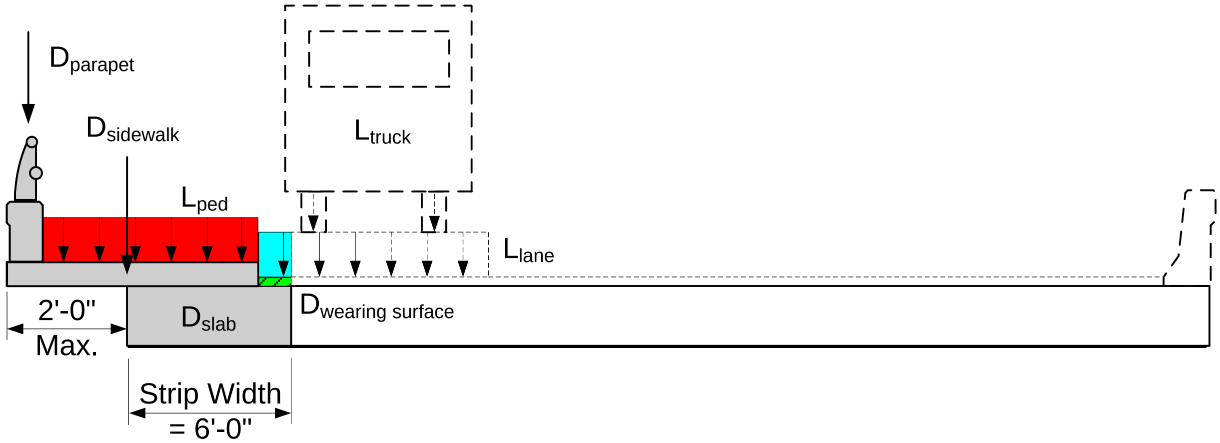
For dead loads, the strip width must resist its self-weight, future wearing surface and all superimposed dead loads. The superimposed dead loads (including parapets, medians, and sidewalk) are distributed uniformly across the entire slab width. The distribution of superimposed dead load to the interior strip width is then computed based on the ratio of the interior strip width to the slab width. Wearing surface is not applied to sidewalks.

For live loads, one lane of live loading is applied to the interior strip width.

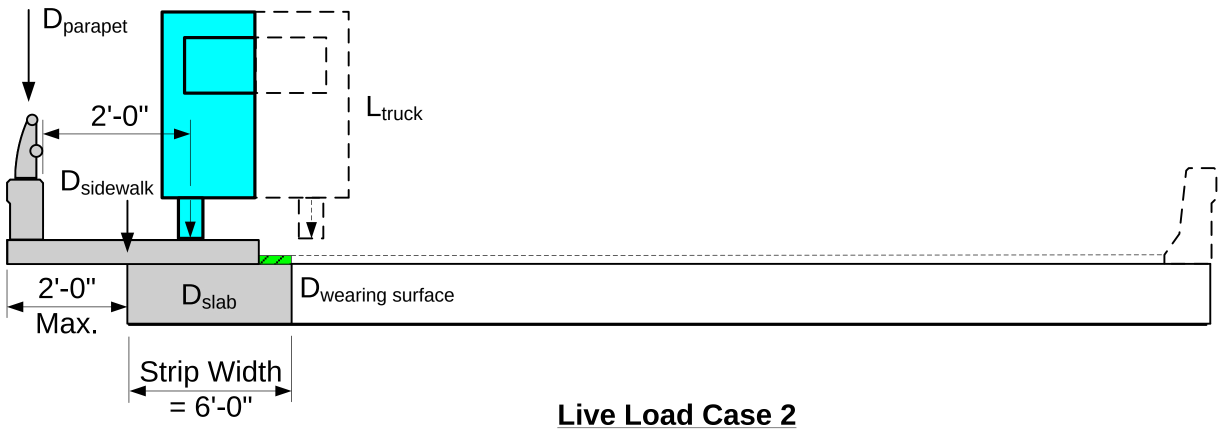
Pedestrian loads are not applied to the interior strip width.

The general equation for loads applied to the interior strip width is as follows:

$$\text{Total Load} = D_{\text{slab}} + D_{\text{wearing surface}} + \left[\left(D_{\text{parapet}-1} + D_{\text{parapet}-2} + D_{\text{sidewalk}} \right) \left(\frac{\text{Strip Width}}{\text{Slab Width}} \right) \right] + \left(L_{\text{truck}} + L_{\text{lane}} \right)$$



Live Load Case 1



Live Load Case 2

Figure 17.2-11
Distribution of Loads to Exterior Strip Width for Slab Structure with Raised Sidewalk

The distribution of loads to the exterior strip is calculated as follows:

For dead loads, the strip width must resist its self-weight and all superimposed dead loads located directly over the strip and on the cantilevered portion of the sidewalk on that side of the bridge. If the overlap of the sidewalk with the slab is > 6'-0", only apply the sidewalk dead load located directly over the exterior strip width and from the cantilevered portion of the sidewalk, to the exterior strip.

Two live load cases shall be considered. For Live Load Case 1, only the portion of the design lane load located directly over the exterior strip width is applied to the exterior strip. The design truck is not applied for Live Load Case 1 due to typical geometry constraints. For Live Load Case 2, the design lane load is not applied. The design truck (see Figure 17.2-1) is placed on the sidewalk with one wheel located 2 feet from the face of the railing. Due to typical geometry constraints, only one wheel is located directly over the exterior strip; therefore, only half of the axle weights (one line of wheels) are applied to the exterior strip.

For pedestrian loads, two load cases shall be considered as described above. For Live Load Case 1, the pedestrian load located directly over the exterior strip and on the cantilevered portion of the sidewalk shall be applied to the exterior strip. For Live Load Case 2, the pedestrian load shall not be applied.

The general equations for loads applied to the exterior strip width are as follows:

For Live Load Case 1:

$$\text{Total Load} = D_{\text{slab}} + \left(D_{\text{wearing surface}} + D_{\text{parapet}} + D_{\text{sidewalk}}\right)_{\text{directly over strip}} + \left(L_{\text{ped}}\right) + \left(L_{\text{lane}}\right)_{\text{directly over strip}}$$

For Live Load Case 2:

$$\text{Total Load} = D_{\text{slab}} + \left(D_{\text{wearing surface}} + D_{\text{parapet}} + D_{\text{sidewalk}}\right)_{\text{directly over strip}} + \left(L_{\text{truck}}\right)_{\text{directly over strip}}$$

17.2.8 Distribution of Loads for Girder Structures

For girder structures, the distribution of dead loads is illustrated in Figure 17.2-12 through Figure 17.2-19. Figure 17.2-12 and Figure 17.2-13 illustrate the distribution of loads for girder structures with no sidewalks. Figure 17.2-14 and Figure 17.2-15 illustrate the distribution of loads for girder structures with sidewalks. Figure 17.2-16 through Figure 17.2-19 illustrate the distribution of loads for girder structures with raised sidewalks. It should be noted that, although medians are not shown in these figures, medians are treated similar to other superimposed dead loads.

For girder structures, distribution of live loads is based on the use of live load distribution factors which are computed as specified in **LRFD [4.6.2.2]** and as summarized in Table 17.2-7. Distribution factors are computed for moment and shear using equations that include girder spacing, span length, deck thickness, the number of girders and the longitudinal stiffness parameter. Separate distribution factors are computed for moment and shear and for interior and exterior girders.

In addition to computing the live load distribution factors, their ranges of applicability should also be checked, as presented in the applicable table in **LRFD [4.6.2.2]**. If the ranges of applicability are not satisfied, then conservative assumptions must be made based on sound engineering judgment.

For girder structures, the most commonly used live load distribution factors are presented in Table 17.2-7.

Application	One Design Lane Loaded	Two or More Design Lanes Loaded
Moment in Interior Girder – **LRFD [Table 4.6.2.2.2b-1]**		
	$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{3}}\right)^{0.1}$	$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{3}}\right)^{0.1}$
	For N_b = 3, use the lesser of the values obtained from the equations above with N_b = 3 or the lever rule.	
Shear in Interior Girder – **LRFD [Table 4.6.2.2.3a-1]**		
	$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$
	For N_b = 3, use the lever rule.	
Moment in Exterior Girder – **LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.2d-1]**		
	Use lever rule	$g=e\cdot g_{interior}$ $e=0.77+\frac{d_e}{9.1}$
		For N_b = 3, use the lesser of the value obtained from the equation above with N_b = 3 or the lever rule.
Shear in Exterior Girder – **LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.3b-1]**		
	Use lever rule	$g=e\cdot g_{interior}$ $e=0.6+\frac{d_e}{10}$
		For N_b = 3, use the lever rule.
Moment Reduction for Skew – **LRFD [Table 4.6.2.2.2e-1]** (not applicable for WisDOT)		
Shear Correction for Skew – **LRFD [Table 4.6.2.2.3c-1]**		

Table 17.2-7
Commonly Used Live Load Distribution Factors for Girder Structures

WisDOT exception to AASHTO:

The rigid cross-section requirement specified in **LRFD [4.6.2.2.2d]** shall not be applied when calculating the distribution factors for exterior girders.

WisDOT exception to AASHTO:

For skewed bridges, WisDOT does not apply skew correction factors for moment reduction, as specified in **LRFD [Table 4.6.2.2.2e-1]**.

WisDOT policy item:

For skewed bridges, WisDOT applies the skew correction factor for shear, as specified in **LRFD [Table 4.6.2.2.3c-1]**, to the *entire span* for *all girders* in a multi-girder bridge.

The following variables are used in Table 17.2-7:

S	=	Spacing of beams (feet)
L	=	Span length (feet)
t_s	=	Depth of concrete slab (inches)
K_g	=	Longitudinal stiffness parameter (inches^4)
N_b	=	Number of beams or girders
g	=	Distribution factor
e	=	Correction factor for distribution
d_e	=	Distance from the exterior web of exterior beam to the interior edge of curb or traffic barrier (feet)

For shear due to live load, in addition to the equations presented in Table 17.2-7, a skew correction factor must be applied in accordance with **LRFD [4.6.2.2.3c-1]**. The skew correction factor equation for shear in girder bridges is as follows:

$$1.0 + 0.20\left(\frac{12.0\,L\,t_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$$

Where:

L	=	Span length (feet)
t_s	=	Depth of concrete slab (inches)
K_g	=	Longitudinal stiffness parameter (inches^4)
θ	=	Skew angle (degrees)

As a general rule of thumb, whenever the live load distribution factors are computed based on the equations presented in *AASHTO LRFD*, the multiple presence factor has already been considered and should not be applied by the engineer. However, when a sketch must be drawn to compute the live load distribution factor, the multiple presence factor must be applied to the computed distribution factor. An example of this principle is in the application of the lever rule.

The multiple presence factor should not be applied to the fatigue limit state for which one design truck is used, regardless of the number of design lanes. However, where the single-lane distribution factor equations are used, as presented in **LRFD [4.6.2.2]** and **LRFD [4.6.2.3]**, the force effects should be divided by 1.20.

For each of the following figures, the distribution of loads for that configuration and girder location is described and a general equation is presented immediately below the corresponding figure. In the general equations, it is assumed that dynamic load allowance is applied to the appropriate live load components.

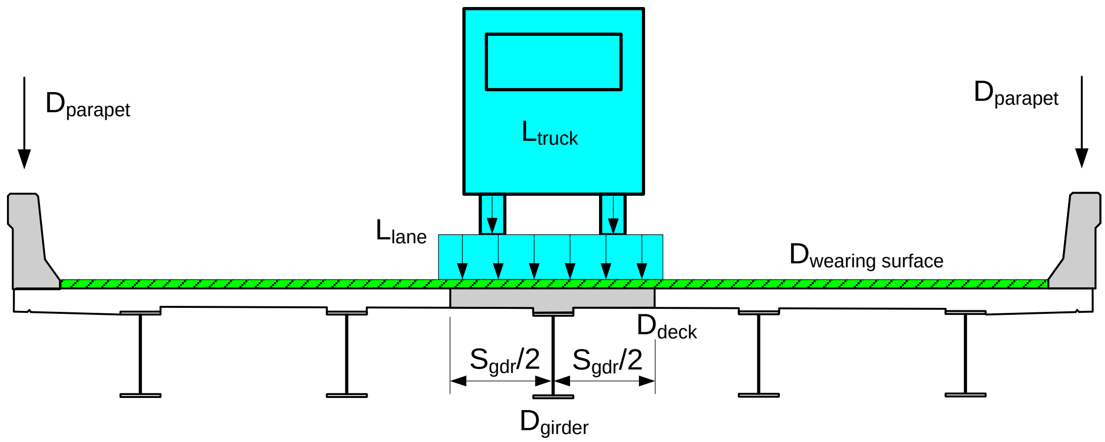


Figure 17.2-12
Distribution of Loads to Interior Girder for Girder Structure

The distribution of loads to the interior girder is calculated as follows:

For dead loads, the interior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch) and all superimposed dead loads. The distribution of the deck weight is based on the girder spacing. The distribution of all superimposed dead loads is based on equal distribution to all girders.

For live loads, the distribution is based on the live load distribution factors (DF) for interior girders presented in Table 17.2-7.

The general equation for loads applied to the interior girder is as follows:

$$\text{Total Load} = D_{girder} + D_{deck} + \left(\frac{D_{wearing\ surface} + 2D_{parapet}}{\text{No. of Girders}} \right) + \left[(DF_{int})(L_{truck} + L_{lane}) \right]$$

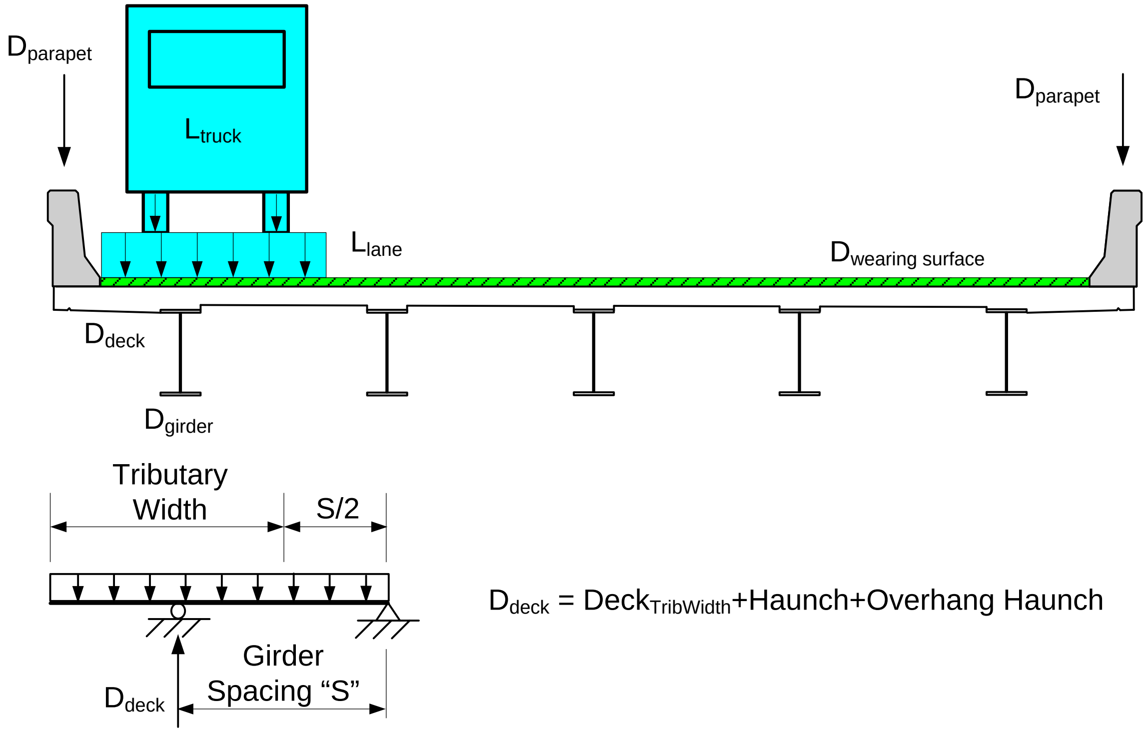


Figure 17.2-13
Distribution of Loads to Exterior Girder for Girder Structure

The distribution of loads to the exterior girder is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight to the exterior girder is based on the tributary width, as shown in the above figure. The distribution of all superimposed dead loads is based on equal distribution to all girders.

For live loads, the distribution is based on the live load distribution factors for exterior girders presented in Table 17.2-7.

The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{girder} + D_{deck} + \left(\frac{D_{wearing\ surface} + 2D_{parapet}}{\text{No. of Girders}}\right) + \left[(DF_{ext})(L_{truck} + L_{lane})\right]$$

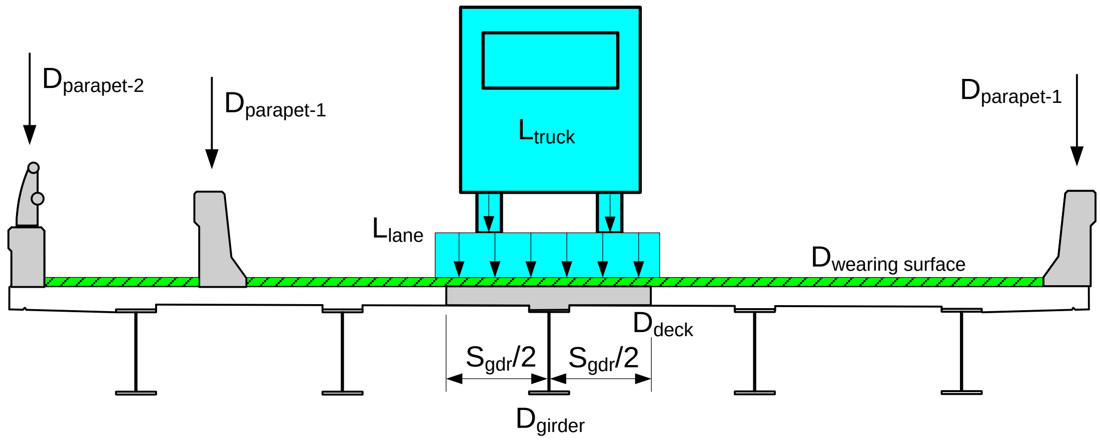


Figure 17.2-14

Distribution of Loads to Interior Girder for Girder Structure with Sidewalk

The distribution of loads to the interior girder is calculated as follows:

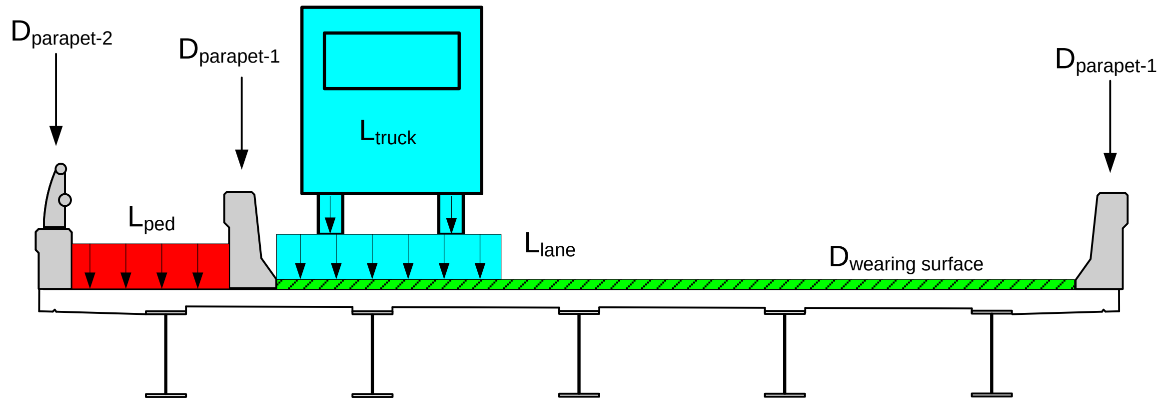
For dead loads, the interior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch) and all superimposed dead loads. The distribution of the deck weight is based on the girder spacing. The distribution of all superimposed dead loads is based on equal distribution to all girders.

For live loads, the distribution is based on the live load distribution factors for interior girders presented in Table 17.2-7.

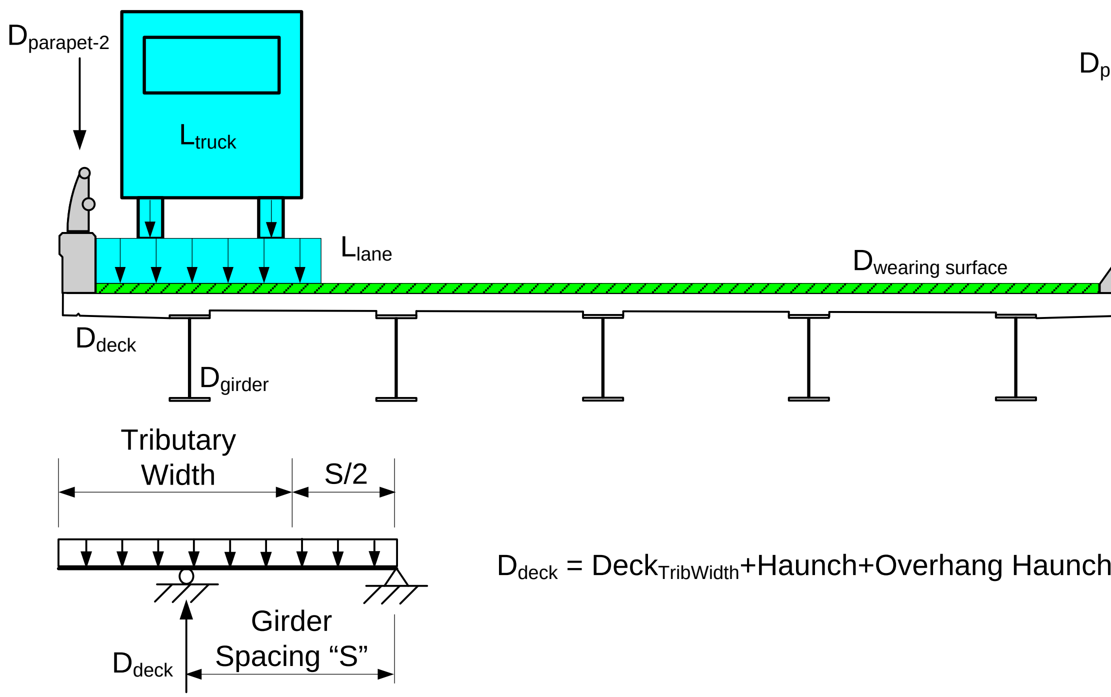
Pedestrian loads are not applied to the interior girder.

The general equation for loads applied to the interior girder is as follows:

$$\text{Total Load} = D_{girderI} + D_{deck} + \left(\frac{D_{wearing\ surface} + 2D_{parapet-1} + D_{parapet-2}}{\text{No. of Girders}} \right) + \left[(DF_{int})(L_{truck} + L_{lane}) \right]$$



Actual Configuration



Use Tributary Width for Deck Load

Design Configuration

Figure 17.2-15

Distribution of Loads to Exterior Girder for Girder Structure with Sidewalk

The distribution of loads to the exterior girder is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight to the exterior girder is based on the tributary width, as shown in the previous figure. The distribution of all superimposed dead loads is based on equal distribution to all girders.

For live loads, it is assumed that the interior parapet is not present. Therefore, the truck and lane are positioned as shown in the Design Configuration portion of the previous figure. The distribution is based on the live load distribution factors for exterior girders presented in Table 17.2-7, assuming the truck and lane as positioned in the Design Configuration portion of the figure.

For pedestrian loads, it is assumed that none are present due to the assumed absence of the interior parapet and the assumed presence of vehicular live load immediately adjacent to the exterior parapet.

The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{\text{girder}} + D_{\text{deck}} + \left(\frac{D_{\text{wearing surface}} + D_{\text{parapet}-1} + D_{\text{parapet}-2}}{\text{No. of Girders}}\right) + \left[\left(DF_{\text{ext}}\right)\left(L_{\text{truck}} + L_{\text{lane}}\right)\right]$$

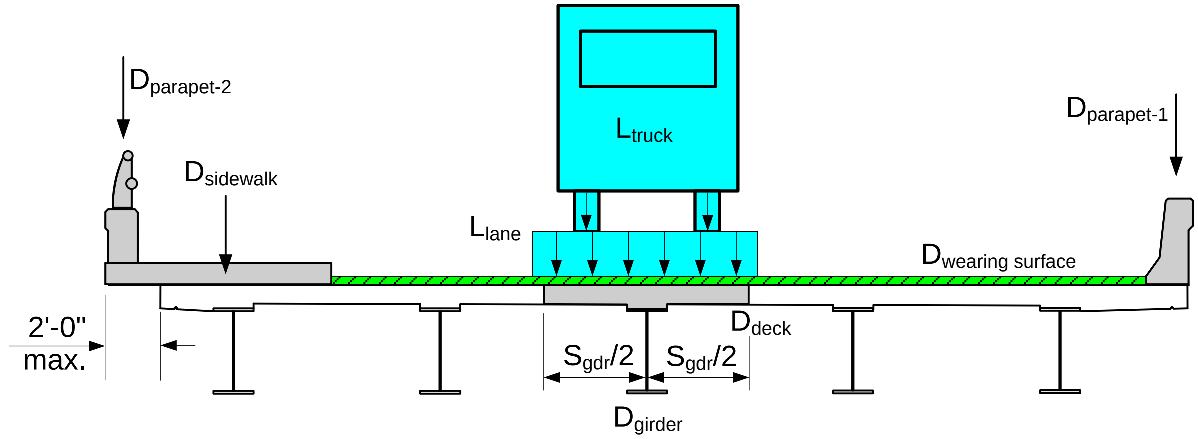


Figure 17.2-16

Distribution of Loads to Interior Girder for Girder Structure with Raised Sidewalk

The distribution of loads to the interior girder is calculated as follows:

For dead loads, the interior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch) and all superimposed dead loads. The distribution of the deck weight is based on the girder spacing. The distribution of all superimposed dead loads is based on equal distribution to all girders.

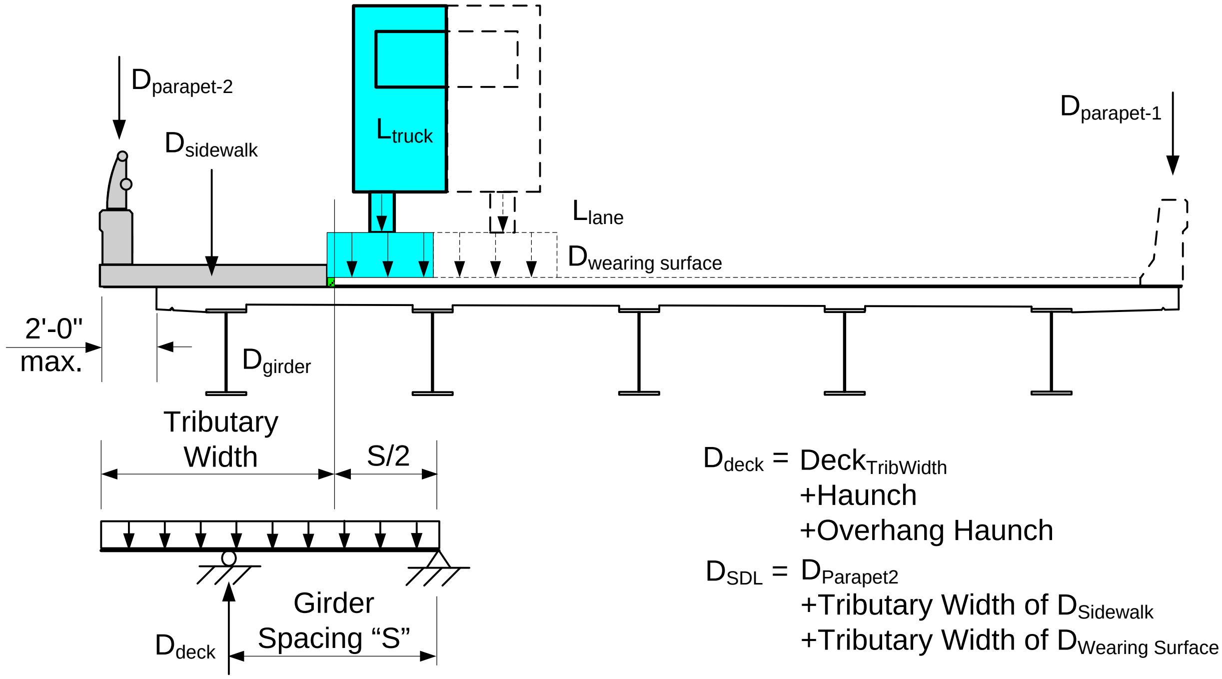
For live loads, the distribution is based on the live load distribution factors for interior girders presented in Table 17.2-7.

Pedestrian loads are not applied to the interior girder.

The general equation for loads applied to the interior girder is as follows:

$$\text{Total Load} =$$

$$D_{\text{girder}} + D_{\text{deck}} + \left(\frac{D_{\text{wearing surface}} + D_{\text{sidewalk}} + D_{\text{parapet-1}} + D_{\text{parapet-2}}}{\text{No. of Girders}} \right) + \left[(DF_{\text{int}})(L_{\text{truck}} + L_{\text{lane}}) \right]$$



Use Tributary Width for Deck and Superimposed Dead Loads

Figure 17.2-17
Distribution of Loads to Exterior Girder for Girder Structure with Raised Sidewalk
Design Case 1

The distribution of loads to the exterior girder for Design Case 1 is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight and all superimposed dead loads to the exterior girder are based on the tributary width, as shown in the previous figure.

For the live load, the live load distribution factor for Design Case 1 is based only on the application of the lever rule. It is recommended for Design Case 1 lane load to use the same distribution factor as for the truck load. The appropriate multiple presence factor of 1.2 must be applied.

The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{girder} + D_{deck} + D_{superimposed\ \ DL} + \left[(DF_{ext})(L_{truck} + L_{lane})\right]$$

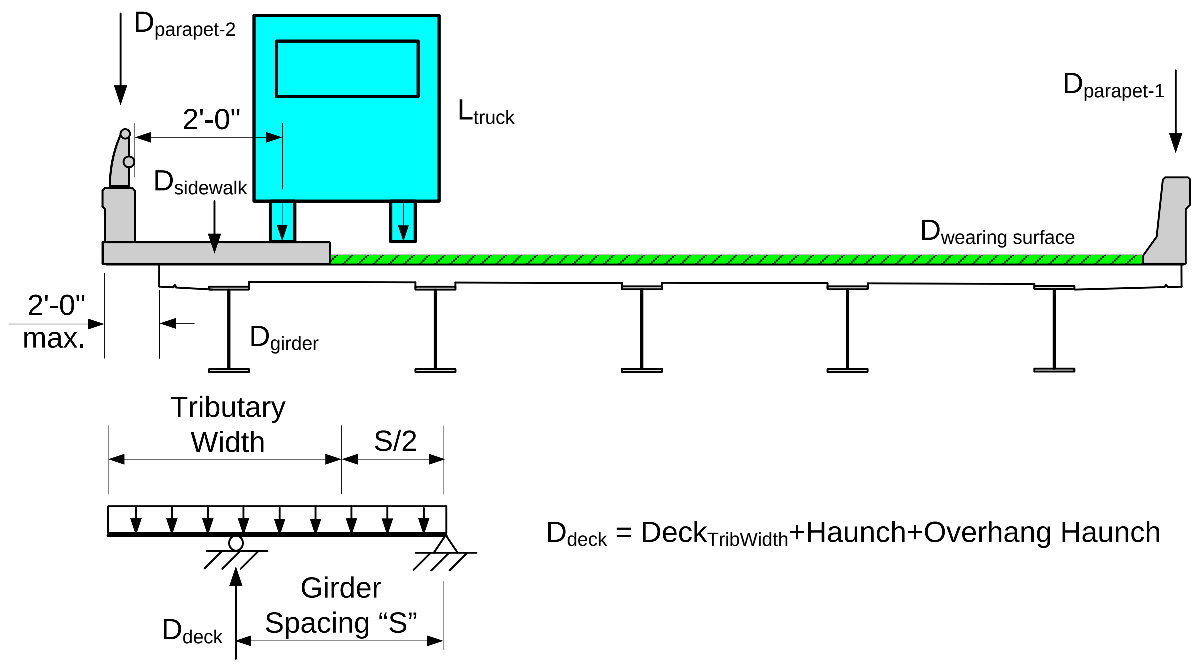


Figure 17.2-18
Distribution of Loads to Exterior Girder for Girder Structure with Raised Sidewalk
Design Case 2

The distribution of loads to the exterior girder for Design Case 2 is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight to the exterior girder is based on the tributary width, as shown in the previous figure. The distribution of all superimposed dead loads is based on equal distribution to all girders.

For the live load, the live load distribution factor for Design Case 2 is based only on the application of the lever rule. The appropriate multiple presence factor of 1.2 must be applied.

The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{\text{girder}} + D_{\text{deck}} + \left(\frac{D_{\text{wearing surface}} + D_{\text{sidewalk}} + D_{\text{parapet-1}} + D_{\text{parapet-2}}}{\text{No. of Girders}}\right) + \left[(DF_{\text{ext}})(L_{\text{truck}})\right]$$

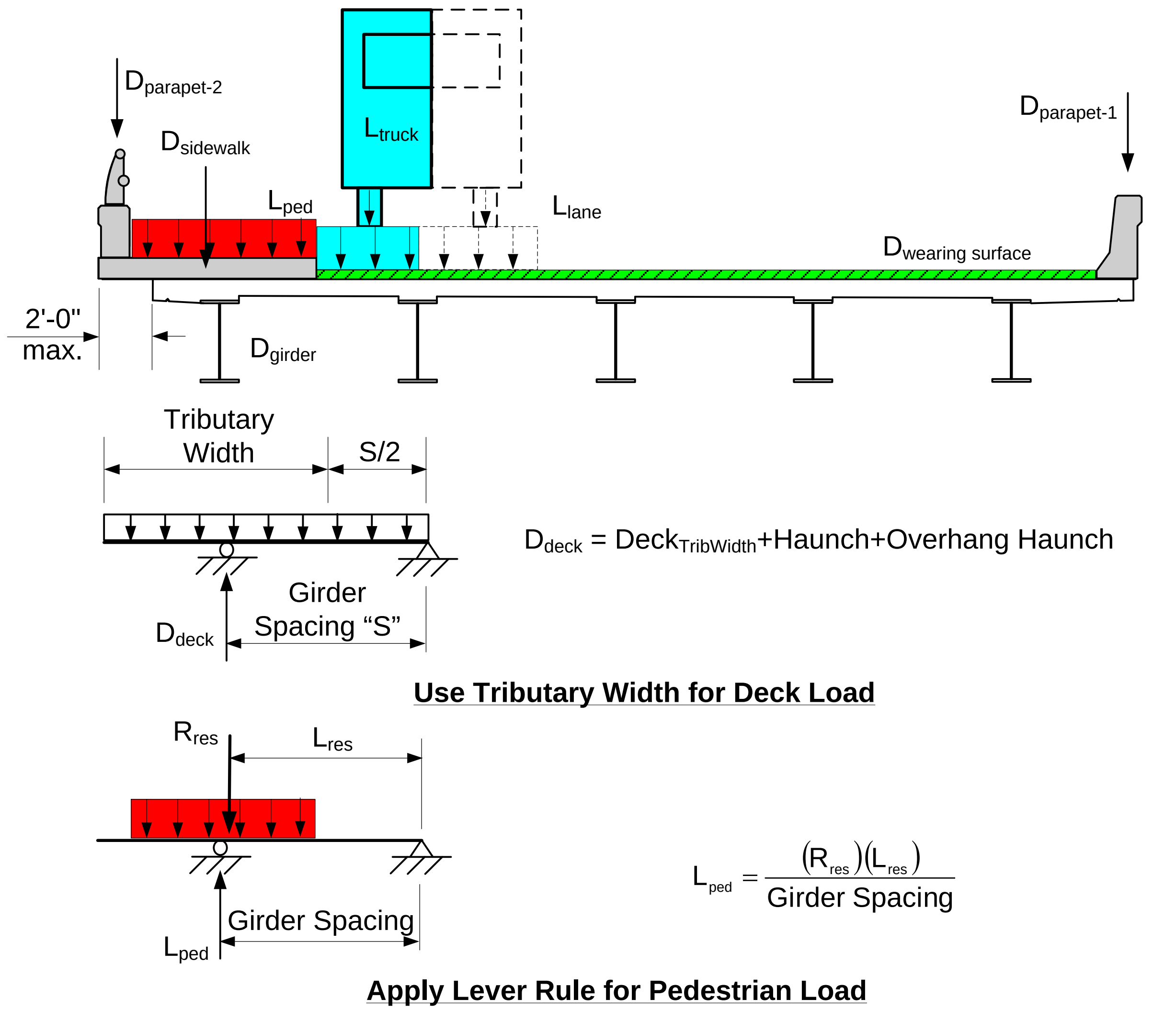


Figure 17.2-19
Distribution of Loads to Exterior Girder for Girder Structure with Raised Sidewalk
Design Case 3

The distribution of loads to the exterior girder for Design Case 3 is calculated as follows:

For dead loads, the exterior girder must resist the girder weight (girder self-weight and any other miscellaneous weight applied to the girder), deck weight (including the concrete haunch and overhang haunch) and all superimposed dead loads. The distribution of the deck weight to the exterior girder is based on the tributary width, as shown in the previous figure. The distribution of all superimposed dead loads is based on equal distribution to all girders.

For the live load, the live load distribution factor for Design Case 3 is based only on the application of the lever rule. It is recommended for Design Case 3 lane load to use the same distribution factor as for the truck load. The appropriate multiple presence factor of 1.0 must be applied.

For pedestrian loads, the distribution to the exterior girder is based on the lever rule, as shown in the previous figure.

The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{\text{girder}} + D_{\text{deck}} + \left(\frac{D_{\text{wearing surface}} + D_{\text{sidewalk}} + D_{\text{parapet}-1} + D_{\text{parapet}-2}}{\text{No. of Girders}} \right) + L_{\text{ped}} + \left[(DF_{\text{ext}})(L_{\text{truck}} + L_{\text{lane}}) \right]$$

17.2.9 Distribution of Dead Load to Substructure Units

For abutment design, the composite dead loads may be distributed equally between all of the girders, or uniformly across the slab.

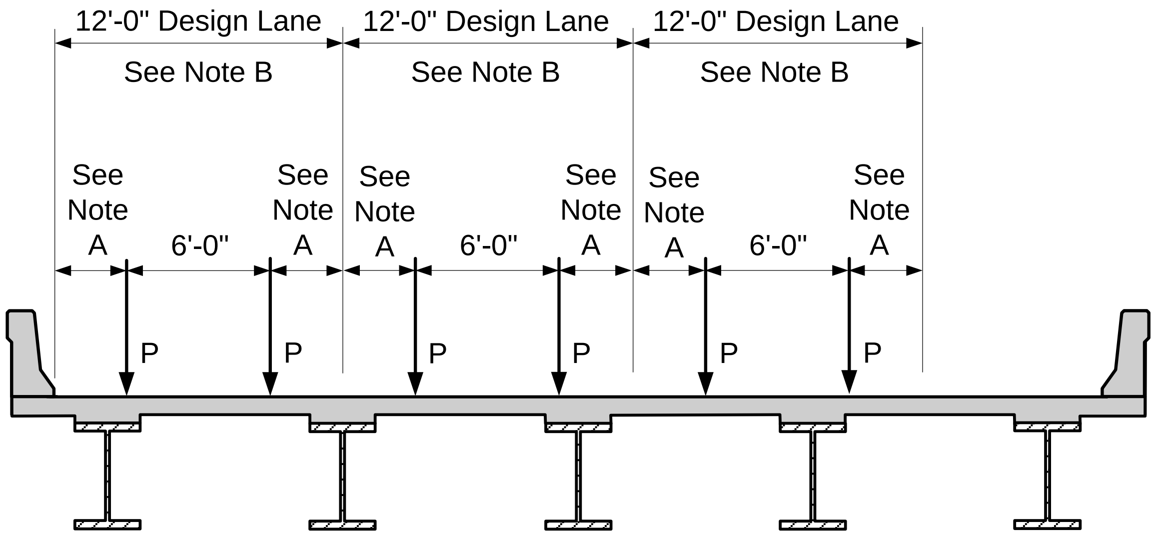
For pier design, the composite dead loads should be distributed equally between all of the girders, or uniformly across the slab, except for bridges with raised sidewalks. For girder bridges with raised sidewalks, follow the aforementioned Design Case 1 & 3 used for exterior girder design. For slab bridges with raised sidewalks, use the loading specified in Live Load Case 1 for exterior strips.

It is acceptable to consider the concrete diaphragm loads to be divided equally between all of the girders and added as point loads to the girder reactions.

17.2.10 Distribution of Live Loads to Substructure Units

See 17.2.9 for additional live load guidance regarding bridges with raised sidewalks. In the transverse direction, the design truck and design tandem should be located in such a way that the effect being considered is maximized. However, the center of any wheel load must not be closer than 2 feet from the edge of the design lane. The transverse live load configuration for a design truck or design tandem is illustrated in Figure 17.2-20. Pedestrian live load may be omitted if trying to maximize positive moment in a multi-columned pier cap.

As a reminder, always be aware to apply loads correctly. For example, for continuous spans the loading to the pier originates from the live load reaction rather than the sum of the live load shears of adjacent spans.



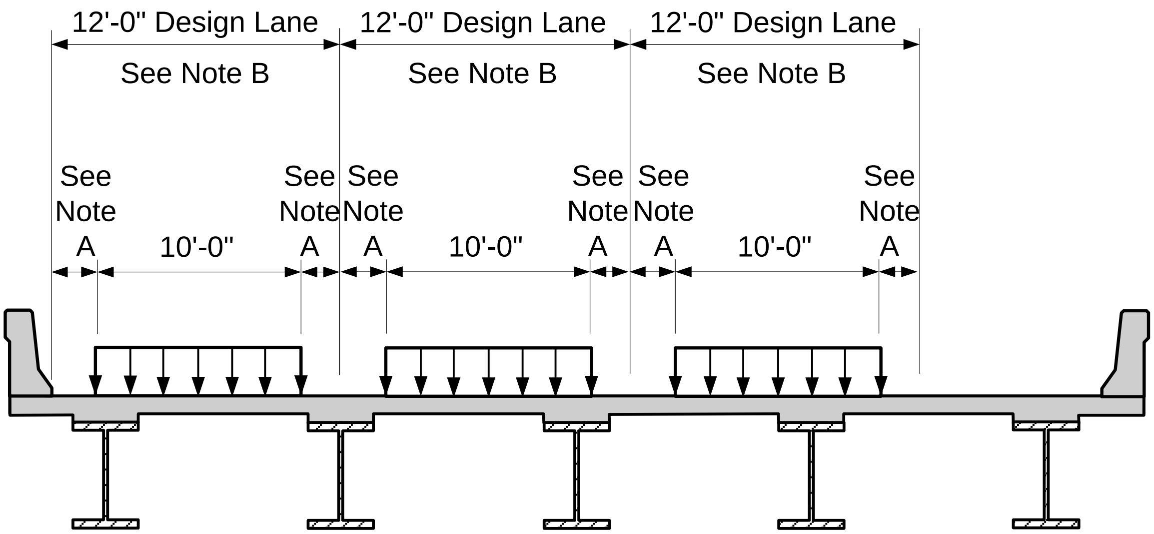
P = Wheel Load

Note A: Position wheel loads within the design lane such that the effect being considered is maximized; minimum = 2'-0".

Note B: Position design lanes across the roadway such that the effect being considered is maximized.

Figure 17.2-20
Transverse Configuration for a Design Truck or Design Tandem

Similarly, the design lane is distributed uniformly over the 10-foot loaded width. Since the design lane is 0.64 kips per linear foot in the longitudinal direction and it acts over a 10-foot width, the design lane load is equivalent to 64 psf. Similar to a design truck or design tandem, the 10-foot loaded width is positioned within the 12-foot design lane such that the effect being considered is maximized, as illustrated in Figure 17.2-21. The 10-foot loaded width may be placed at the edge of the 12-foot design lane.



Note A: Position 10'-0" lane loads within the 12'-0" design lane such that the effect being considered is maximized; minimum = 0'.

Note B: Position 12'-0" design lanes across the roadway such that the effect being considered is maximized.

Figure 17.2-21
Transverse Configuration for a Design Lane

When live load reactions are calculated at substructure units different methods of distributing the loads are used for the axles on the substructure and for the axles in the spans. The load to a girder for an axle directly over the substructure unit is based on simple beam distribution between the girders. The reactions for the axles located within the span are based on the shear distribution factors.

WisDOT policy item:

A 10 foot design lane width may be used for the distribution of live loads to a pier cap.

For use in design of the foundations, the live load reactions should be tracked for both the Strength and Service load cases, as well as with and without the dynamic load allowance (IM). Note that the IM is not applied to the lane load portion of the live load reaction, so the reaction without the IM cannot be factored out of the reaction with IM.

17.2.11 Composite Section Properties

The effective flange width is the width of the deck slab that is to be taken as effective in composite action for determining resistance for all limit states. In accordance with **LRFD [4.6.2.6]**, the composite section width is taken as the tributary width perpendicular to the axis of the girder.

For exterior beams, the effective flange width is taken as one-half the effective width of the adjacent interior beam, plus the width of the overhang.

17.2.12 Allowable Live Load Deflection

WisDOT policy item:

LRFD [2.5.2.6.2] specifies optional live load deflection criteria for simple or continuous spans. However, the deflection criteria presented in Table 17.2-8 is required by WisDOT

Structure Type	Allowable Live Load Deflection
Conventional girder structure without pedestrians	L/800
Conventional girder structure with pedestrians	L/1000
Concrete slab structure	L/1200

Table 17.2-8
Allowable Live Load Deflection

17.2.13 Actual Live Load Deflection

The distribution factor for computing live load deflection is not the same as the moment distribution factor, because it is assumed that for straight bridges all beams or girders act together and have an equal deflection. However, for curved bridges, each girder must be checked individually.

For steel girder structures, composite section properties for deflection computations should be based on n rather than 3n. For concrete slab structures, the deflections should be based on the entire slab width acting as a unit, using the gross moment of inertia, I_g.

Using an analysis computer program, the maximum live load deflection can be computed as follows:

All design lanes are loaded and the appropriate multiple presence factor is used.

For composite design, the design cross section includes the entire width of the deck. As specified in AASHTO LRFD, the barriers and sidewalks may be included in the stiffness computations. However, due to the complexity of such computations, this should not be standard practice for WisDOT structures.

The number and position of loaded lanes is selected to provide the worst effect.

The live load portion of Service I limit state is used.

Dynamic load allowance is included.

The live load is taken as live load combinations LL#5: Design Truck + IM or LL#6: 25%(Design Truck + IM)+ Lane from 17.2.4.2.6.

17.3 Selection of Structure Type

The selection of the proposed structure type is determined from evaluation of the Structure Survey Report with accompanying supplemental data, current construction costs and preference based on past experience. In selecting the most economical structure, ease of fabrication and erection, general features of terrain, roadway geometrics, subsurface exploration and geographic location in the State of Wisconsin are considered. The proposed structure must blend into existing site conditions in a manner that does not detract from its surrounding environment. Every attempt should be made to select an aesthetically attractive structure consistent with structural requirements, economy and geographic surroundings. For information about bridge aesthetics, see Chapter 4 – Aesthetics.

The economical span ranges of various types of structures are given in Chapter 5 – Economics and Costs. Superstructure span lengths are related to the cost of the substructure units. If the substructure units are relatively expensive, it is generally more economical to use longer span lengths available for a given structure type. Practicality dictates using the average structure length for twin structures if the preliminary structure lengths are within approximately 3 feet. In addition, a multiple-span structure should be made symmetrical if its end spans are within approximately 3 feet in length of each other.

All structure span lengths are rounded off to the nearest 1 foot, except for stream crossings and multi-span prestressed girder structures where the span lengths are adjusted to maintain equal girder lengths. For example, a typical multiple-span prestressed girder structure has interior spans longer than its exterior spans. Refer to the Standard Details for Pretensioned Girders, Slab and Superstructure Details for details of girder lengths at abutment and piers.

At stream crossings, structure span lengths should be designed to the nearest foot (center-to-center bearing) and skew angles should be designed in multiples of 5°. This results in standardized span lengths and skew angles.

For geometric considerations in structure selection, reference is made to Chapter 3 – Design Criteria. The requirements for structure expansion and fixed pier locations are presented in Chapter 12 – Abutments, and bearing types are described in Chapter 27 – Bearings. Expansion joint types and requirements are specified in Chapter 28 – Expansion Devices. Since the skew angle for most snow plow blades is 35°, it is desirable to avoid this skew angle for bridge joints. This reduces the chances of joint damage resulting from the plow blades dropping into the expansion joints.

Use of non-redundant structures, including single-box and two-box steel box girder bridges, should be avoided unless absolutely necessary. Certain situations, including extreme span length over a navigational channel or tight curvature, may necessitate such bridges.

17.3.1 Alternate Structure Types

When developing bridge plans, consider the following procedures:

- Base preliminary plan development on an engineering and economic evaluation of alternate designs.

- Evaluate alternative designs on the basis of competitive materials appropriate to a specific structure type.

- Do not propose specific construction methods or erection procedures in the plans unless constraints are necessary to meet specific project requirements.

- Make an economic evaluation of preliminary estimates based on state-of-the-art methods of construction for structure types.

- Consider future structure maintenance needs in the structure's design in order to provide life-cycle costing data.

- Consider alternate plans where experience, expertise and knowledge of conditions clearly indicate that they are justified. Alternate plans are not compatible with stage construction and should not be used in these situations.

- Value engineering concepts are recognized as being cost effective. Apply these concepts to the selection of structure type, size and location throughout the plan development process.

17.4 Superstructure Types

Superstructures are classified as deck or through types.

For deck type structures, the roadway is above or on top of the supporting structures. Examples of deck type structures are girder bridges and steel deck-trusses.

For through type structures, the roadway passes between two elements of the superstructure. Examples of through type structures are steel through-trusses and tied-arch bridges.

Through type structures are generally used where long span lengths are required. Deck type structures are more common, because they lend themselves to future widening if increased traffic requires it.

Some of the various types of superstructures used in Wisconsin are as follows:

1. Concrete slab (flat and haunched)

   Concrete slab structures are adaptable to roadways with a high degree of horizontal curvature. This superstructure type is functional for short to medium span lengths and is relatively economical to construct and maintain. The practical range of span lengths for concrete slab structures can be increased by using haunched slab structures.

WisDOT policy item:

Concrete slab structures are limited to sites requiring a skew angle of 30 degrees or less.

   Voided slab structures are not currently being used due to excessive longitudinal cracking over the voids in the negative moment region. For more information about concrete slab structures, refer to Chapter 18 – Concrete Slab Structures.

2. Prestressed concrete girder

   Prestressed concrete girder structures are very competitive from a first cost standpoint and require very little maintenance. Prestressed concrete girders are produced by a fabrication plant certified by WisDOT. Future widening can be accomplished with relative ease. For more information about prestressed concrete girder bridges, refer to Chapter 19 – Prestressed Concrete.

3. Concrete T-beam

WisDOT policy item:

The concrete T-beam has had limited use in Wisconsin during recent years and is no longer used.

4. Prestressed box girder

   Prestressed box girder structures have the advantage of rapid construction where traffic must be diverted. Elimination of the need for falsework is a particular advantage when vertical clearances are critical during the construction phase. Experience indicates that, from a first-cost standpoint, these structures are more expensive to construct than concrete slab structures. For more information, refer to Chapter 19 – Prestressed Concrete.

5. Concrete box girder

   The concrete box girder structure is aesthetically adaptable for urban sites having roadways with a high degree of horizontal curvature or large skew angles. This structure is frequently employed in multi-level interchanges where horizontal clearances are limited, since the pier cap is an integral part of the superstructure. However, problems can be encountered in maintenance with deck replacements requiring shoring.

6. Concrete rigid frame

   The concrete rigid frame is more costly than other superstructure types. However, the concrete rigid frame is known for its aesthetic value and is used primarily in public parks and urban areas where the span lengths are similar to concrete slab structures and where approach embankments are relatively high.

7. Steel rolled section and welded plate girder

   Welded plate girders are less expensive than rolled sections with cover plates because of their reduced allowable design stress resulting from the fatigue criteria. Welded plate girders have greater versatility in allowing variable web thickness and depth, as well as variable flange thicknesses. Future widening can be accomplished with relative ease. For more information, refer to Chapter 24 – Steel Girder Structures and Chapter 38 – Railroad Structures.

8. Steel box girder

   Steel box girder structures have span length capabilities similar to plate girders. Aesthetically, they present a smooth, uncluttered appearance due to their closed box sections. Current experience reveals that steel box girders require more material than conventional steel plate girders. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

9. Steel tied arch and steel truss

   Unusual bridge sites, such as major river and harbor crossings, may require the use of longer span lengths than conventional deck type superstructures can accommodate. For such conditions, a steel tied arch or a steel truss can be used effectively.

10. Timber longitudinally laminated decks

Timber structures blend well in natural settings and are relatively easy to construct with light construction equipment. Timber longitudinally laminated deck structures have low profiles that generally provide large clearances for high water. Their application is limited by the range of span lengths and economics in comparison to concrete slabs. For more information, refer to Chapter 23 – Timber Structures.

17.5 Design of Slab on Girders

17.5.1 General

The design of concrete decks on prestressed concrete or steel girders is based on **LRFD [4.6.2.1]**. Moments from truck wheel loads (one or two trucks side by side) are distributed over a width of deck which spans perpendicular to the girder. The width of deck or width of equivalent strip is presented in **LRFD [Table 4.6.2.1.3-1]**. Positive moments are distributed over a different deck width than negative moments. The distribution width in inches is equal to 26.0 + 6.6 S for positive moments and 48.0 + 3.0 S for negative moments, where S equals girder spacing in feet.

To minimize transverse deck cracking, a minimum slab thickness of 8 inches is used for all decks on new bridges. For deck replacements, a thinner deck may be used if a reduced dead load is required to increase live load capacity. Research on transverse deck cracking (*NCHRP Report 297*) recommends smaller diameter reinforcement to reduce transverse deck cracking. The maximum size of transverse bars used is #5, with a minimum spacing of 6 inches. Identical bar size and spacing is used for the top and bottom transverse bars with each layer offset half the bar spacing from the other. If top and bottom transverse bars align, they form a weakened section within the concrete that is more susceptible to cracking.

For bridges with deck slabs on girders, the most economical structure can often be achieved by using as few lines of girders as possible. However, for prestressed concrete girders, it is often more economical to add an extra girder line than to use debonded strands with the minimum number of girder lines. After the number of girders has been determined, adjustments in girder spacing should be investigated to see if slab thickness can be minimized.

17.5.2 Two-Course Deck Construction

WisDOT policy item:

The use of two-course deck construction should be avoided and its use requires BOS approval.

For skews of 20 degrees or greater, the machine used to strike off and finish the concrete must have its longitudinal axis within 20 degrees of the centerline of bearing of the substructure units. This produces more equal girder loads in a span during the concrete pour, which results in dead load deflections being closer to the theoretical computed deflections.

However, for steel girders with wide decks and large skews or for continuous long-span steel girders, final dead load deflections may not be within a reasonable allowable variance from the theoretical. By using two-course construction, any discrepancies in deflections in the first pour can be corrected by varying the thickness of the second pour since most of the deflection will occur during the first pour.

When using two-course construction, the first pour is 1 inch less in thickness (1.5” bar cover) than the standard deck thickness and the second pour is a 2” minimum thickness Class E concrete overlay. For two-course deck construction, an additional 20 psf should be added for a future wearing surface. The top surface of the first pour is given a dragged or broom finish to obtain a roughened surface.

A report by the Kansas DOT entitled “Cracking and Chloride Content in Reinforced Concrete Bridge Decks” (Report No. K-Tran: KU-01-9) has determined that two-course deck construction results in decks that have more severe cracking than monolithic decks. The report also states that the average chloride concentration at crack locations exceeds the corrosion threshold by the end of the first winter season after construction. Some agencies specify a high density second course concrete overlay to provide a more durable riding surface.

17.5.3 Reinforcing Steel for Deck Slabs on Girders

The following sections describe the design requirements for reinforcing steel for deck slabs on girders. Design tables are included that can be used for common superstructure configurations.

17.5.3.1 Transverse Reinforcement

The live load moments used to determine the size and spacing of the transverse bars are presented in **LRFD [Table A4-1]**. This table presents positive and negative live load moments per unit width, in units of kip-feet per foot. Moments are given for girder spacings ranging from 4’-0” to 15’-0” in increments of 3”. Negative moments are presented for varying distances from the centerline of girder to the design section.

The negative dead load moment over the support is determined from the following equation:

$$M_{DL} = \frac{W\,S^2}{10}$$

Where:

W	=	Uniform dead load of slab and wearing surface
S	=	Girder spacing

The positive dead load moment is determined using the following equation:

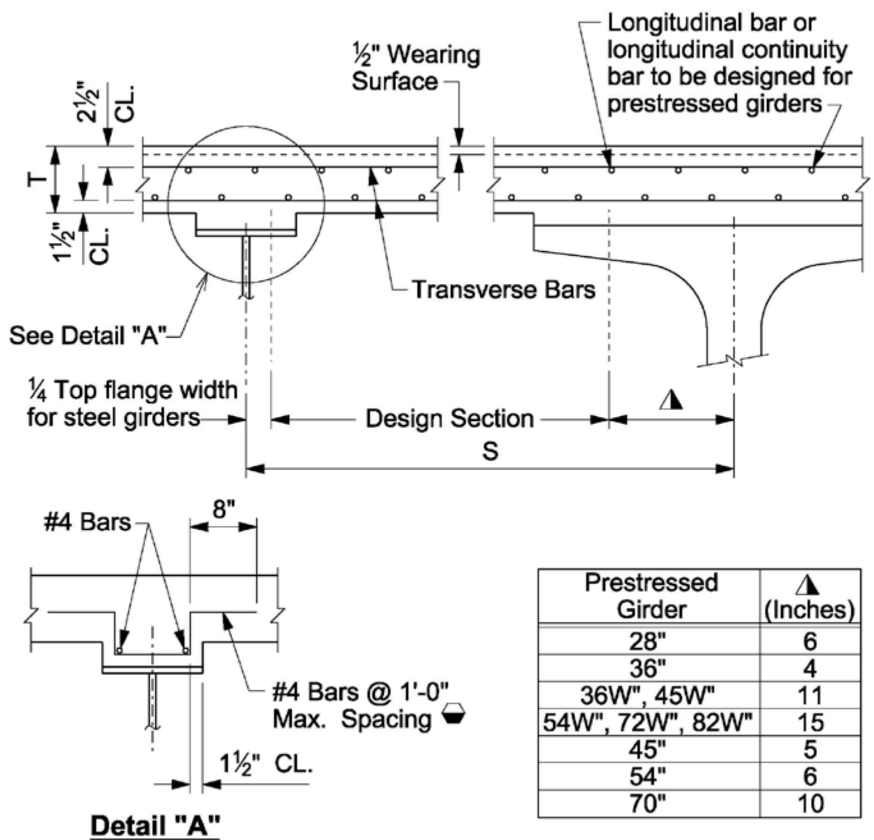
$$M_{DL} = \frac{W\,S^2}{12.5}$$

WisDOT's current practice is to ignore the moments in the deck (not including in the deck overhang) resulting from superimposed dead loads (such as parapets and medians).

Negative moments at supports are adjusted to equal the moment at the location of the design section being considered.

The distance from the centerline of the girder to the design section is computed in accordance with **LRFD [4.6.2.1.6]**. For steel beams, this distance is equal to one-quarter of the flange width from the centerline of support. For prestressed concrete girders, this distance is equal to the values presented in Figure 17.5-1, along with bar locations and clearances.

Note: Transverse reinforcing steel requirements (bar size and spacing) are determined for both positive moment requirements and negative moment requirements, and the same reinforcing steel is used in both the top and bottom of slab as shown in Table 17.5-1 and Table 17.5-2. Longitudinal reinforcement in Table 17.5-3 and Table 17.5-4 is based on a percentage of the bottom transverse reinforcement required *by actual design calculations* (not a percentage of what is in the tables). **The tables should be used for deck reinforcement, with continuity bars in prestressed girder bridges being the only deck reinforcement requiring calculation.**



Prestressed Girder	(Inches)
28"	6
36"	4
36W", 45W"	11
54W", 72W", 82W"	15
45"	5
54"	6
70"	10

Where the spacing of the existing shear connectors is equal to or less than 1'-0", spacing of #4 hat bars to match.
Where the spacing of the existing shear connectors is more than 1'-0", spacing of #4 hat bars to be 1'-0".

Figure 17.5-1
Transverse Section thru Slab on Girders

For skews of 20° and under, place transverse bars along the skew. For skews greater than 20°, place transverse bars perpendicular to the girders.

Detail "A", as presented in Figure 17.5-1, should be used for decks when shear connectors extend less than 2 inches into the slab on steel girder bridges or 3 inches on prestressed concrete girder bridges.

Several transverse reinforcing steel tables are provided in this chapter. The reinforcing steel in Table 17.5-1 and Table 17.5-2 does not account for deck overhangs. However, the minimum amount of reinforcing steel required in the deck overhangs is presented in various design tables in 17.6.

The reinforcement shown in Table 17.5-1 and Table 17.5-2 is based on both the Strength I requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.7.3.4]**. The bar spacing cannot exceed the value from the following formula:

$$s \le \frac{700(\gamma)}{\beta_s f_s} - 2 d_c$$

Where:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

γ	=	0.75 for decks
β_s	=	Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face
f_s	=	Tensile stress in reinforcement at the service limit state (ksi) ≤ 0.6 f_y
d_c	=	Top concrete cover less ½ inch wearing surface plus ½ bar diameter or bottom concrete cover plus ½ bar diameter (inches)
h	=	Slab depth minus ½ inch wearing surface (inches)

<u>WisDOT policy item:</u>

The thickness of the sacrificial ½-inch wearing surface shall not be included in the calculation of d_c.

Table 17.5-1 and Table 17.5-2 were developed for specified values of the distance from the centerline of girder to the design section for negative moment. Those specified values – 0, 3, 6, 9, 12 and 18 inches – were selected to match values used in **AASHTO [Table A4-1]**. For a girder in which the distance from the centerline of girder to the design section for negative moment is not included in Table 17.5-1 and Table 17.5-2, the engineer may interpolate between the closest two values in the tables or can use the more conservative of the two values.

Transverse Reinforcing Steel for Deck Slabs on Girders for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness "T" ≥ 8"							
Slab Thickness "T" (Inches)	Girder Spacing "S"	Distance from Centerline of Girder to Design Section					
		0"	3"	6"	9"	12"	18"
8	4'-6"	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	4'-9"	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-0"	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-3"	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-6"	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7	#4 @ 7
8	5'-9"	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7	#4 @ 7
8	6'-0"	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	6'-3"	#5 @ 7	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	6'-6"	#5 @ 7	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	6'-9"	#5 @ 7	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
8	7'-0"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8	7'-3"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8	7'-6"	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8	7'-9"	#5 @ 6	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8	8'-0"	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8.5	8'-3"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8.5	8'-6"	#5 @ 6.5	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
8.5	8'-9"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8.5	9'-0"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
8.5	9'-3"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
9	9'-6"	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
9	9'-9"	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
9	10'-0"	#5 @ 6	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
9	10'-3"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
9	10'-6"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
9.5	10'-9"	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
9.5	11'-0"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8
9.5	11'-3"	#6 @ 7	#5 @ 6	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
9.5	11'-6"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10	11'-9"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8

10	12'-0"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8
10	12'-3"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10	12'-6"	#6 @ 7	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10.5	12'-9"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 8
10.5	13'-0"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
10.5	13'-3"	#6 @ 7	#6 @ 7	#5 @ 6.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5
10.5	13'-6"	#6 @ 6.5	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5
11	13'-9"	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5
11	14'-0"	#6 @ 7	#6 @ 7	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5

Table 17.5-1
Transverse Reinforcing Steel for Deck Slabs on Girders
for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness "T" ≥ 8"

Transverse Reinforcing Steel for Deck Slabs on Girders for Deck Replacements, HL-93 Loading, Slab Thickness "T" < 8"							
Slab Thickness "T" (Inches)	Girder Spacing "S"	Distance from Centerline of Girder to Design Section					
		0"	3"	6"	9"	12"	18"
6.5	4'-0"	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	4'-3"	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	4'-6"	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	4'-9"	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	5'-0"	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
6.5	5'-3"	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	5'-6"	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	5'-9"	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-0"	#6 @ 6.5	#6 @ 7.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-3"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-6"	#6 @ 6	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
6.5	6'-9"	(1)	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7
6.5	7'-0"	(1)	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5
7	4'-0"	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	4'-3"	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	4'-6"	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	4'-9"	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8

7	5'-0"	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	5'-3"	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	5'-6"	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	5'-9"	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7	6'-0"	#6 @ 7.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	6'-3"	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	6'-6"	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	6'-9"	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	7'-0"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7	7'-3"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7	7'-6"	#6 @ 6	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7	7'-9"	(1)	#6 @ 6.5	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7
7	8'-0"	(1)	#6 @ 6.5	#5 @ 6	#5 @ 7	#5 @ 7	#5 @ 7
7.5	4'-0"	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	4'-3"	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	4'-6"	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	4'-9"	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5'-0"	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5'-3"	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5'-6"	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5
7.5	5'-9"	#5 @ 7	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
7.5	6'-0"	#5 @ 7	#5 @ 8	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
7.5	6'-3"	#5 @ 6.5	#5 @ 7.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5	#5 @ 8.5
7.5	6'-6"	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7.5	6'-9"	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8	#5 @ 8
7.5	7'-0"	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8
7.5	7'-3"	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 8	#5 @ 8	#5 @ 8
7.5	7'-6"	#6 @ 7	#5 @ 6.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	7'-9"	#6 @ 7	#5 @ 6	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	8'-0"	#6 @ 6.5	#6 @ 7.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	8'-3"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5
7.5	8'-6"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7.5	8'-9"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7.5	9'-0"	#6 @ 6.5	#6 @ 7.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7
7.5	9'-3"	#6 @ 6.5	#6 @ 7	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5

7.5	9'-6"	#6 @ 6	#6 @ 6.5	#5 @ 6	#5 @ 6.5	#5 @ 6.5	#5 @ 6.5

(1) When these regions are encountered, the next thicker deck section shall be used.

Table 17.5-2
Transverse Reinforcing Steel for Deck Slabs on
Girders for Deck Replacements, HL-93 Loading, Slab Thickness "T" < 8"
(Only use Table 17.5-2 if Bridge Rating is unacceptable with "T" ≥ 8")

The transverse reinforcing steel presented in Table 17.5-1 and Table 17.5-2 is designed in accordance with *AASHTO LRFD*. The tables are developed based on deck concrete with a 28-day compressive strength of f'_c = 4 ksi and reinforcing steel with a yield strength of f_y = 60 ksi. However, the same tables should be used for concrete strength of 5 ksi.

The clearance for the top steel is 2 1/2", and the clearance for the bottom steel is 1 1/2". The dead load includes 20 psf for future wearing surface.

The reinforcing bars shown in the tables are for one layer only. Identical steel should be placed in both the top and bottom layers.

17.5.3.2 Longitudinal Reinforcement

The amount of bottom longitudinal reinforcement required is as specified in **LRFD [9.7.3.2]** and shown in Table 17.5-3 and Table 17.5-4. It is based on a percentage of the transverse reinforcing steel for positive moment. For the main reinforcement perpendicular to traffic, the percentage equals:

$$\frac{220}{\sqrt{S}} \leq 67\%$$

Where:

S = Girder spacing, as calculated based on Figure 17.5-1 (feet)

<u>WisDOT exception to AASHTO:</u>

The girder spacing shall be used in the equation above for calculating the percentage of transverse steel to be used as longitudinal reinforcement. This definition replaces the one stated in **LRFD [9.7.3.2]** to use the effective girder spacing.

The minimum amount of longitudinal reinforcement required for temperature and shrinkage in each of the top and bottom layers is given by **LRFD [5.10.8]** as follows:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$

and

$0.11 \le A_s \le 0.60$

Where:

A_s	=	Area of reinforcement in each direction and each face (in.^{2}/ft.)
f_y	=	Reinforcing steel yield strength = 60 ksi
b	=	Width of deck (inches)
h	=	Thickness of deck (inches)

In addition, the minimum amount of longitudinal steel in both layers used by WisDOT is #4 bars at 9" spacing to reduce transverse deck cracking. Identical amounts of steel are placed in both the top and bottom layer, and the reinforcing bars are uniformly spaced from edge to edge of slab. Table 17.5-3 and Table 17.5-4 use the same longitudinal bar spacings throughout a given bridge deck.

See Chapter 19 – Prestressed Concrete for design guidance regarding continuity reinforcement for prestressed girder bridges.

When continuous steel girders are not designed for negative composite action, **LRFD [6.10.1.7]** requires an area of longitudinal steel in both the top and bottom layer equal to 1% of the cross-sectional area of the slab in the span negative moment regions. The "d" value used for this computation is the total slab thickness excluding the wearing surface. This reinforcing steel is uniformly spaced from edge to edge of slab in the top and bottom layer. It is required that two-thirds of this reinforcement be placed in the top layer. The values shown in Table 17.5-3 and Table 17.5-4 provide adequate reinforcement to cover the requirements of **LRFD [6.10.1.7]**. It is WisDOT practice to abide by **LRFD [6.10.1.7]** for new bridges utilizing negative composite action, as well. See 24.7.6 for determining continuity bar cutoff locations for new bridges and rehabilitation bridges.

Longitudinal Reinforcing Steel for Deck Slabs on Girders for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness "T" ≥ 8"			
		Bar Size and Spacing (Inches)	
		Prestressed Girder Bridges	Steel Girder Bridges
Slab Thickness "T" (Inches)	Girder Spacing "S"	#4's Top and Bottom, Continuity Reinforcement To Be Designed (Top)	#4's Top and Bottom, Continuity Reinforcement** #6's (Top)
8	4'-6"	9.0	8.5
8	4'-9"	9.0	8.5
8	5'-0"	9.0	8.5
8	5'-3"	9.0	8.5
8	5'-6"	9.0	8.5
8	5'-9"	9.0	8.5
8	6'-0"	9.0	8.5

8	6'-3"	9.0	8.5
8	6'-6"	9.0	8.5
8	6'-9"	9.0	8.5
8	7'-0"	9.0	8.5
8	7'-3"	9.0	8.5
8	7'-6"	8.5	8.5
8	7'-9"	8.5	8.5
8	8'-0"	8.0	8.5
8.5	8'-3"	9.0	8.0
8.5	8'-6"	8.5	8.0
8.5	8'-9"	8.5	8.0
8.5	9'-0"	8.5	8.0
8.5	9'-3"	8.0	8.0
9	9'-6"	9.0	7.5
9	9'-9"	8.5	7.5
9	10'-0"	8.5	7.5
9	10'-3"	8.0	7.5
9	10'-6"	8.0	7.5
9.5	10'-9"	8.0	7.0
9.5	11'-0"	8.0	7.0
9.5	11'-3"	8.0	7.0
9.5	11'-6"	8.0	7.0
10	11'-9"	8.0	6.5
10	12'-0"	8.0	6.5
10	12'-3"	8.0	6.5
10	12'-6"	8.0	6.5
10.5	12'-9"	8.5	6.0
10.5	13'-0"	8.0	6.0
10.5	13'-3"	8.0	6.0
10.5	13'-6"	8.0	6.0
11	13'-9"	8.0	6.0
11	14'-0"	8.0	6.0

Legend:

** Use for deck slabs on steel girders in negative moment regions. New bridge shall be designed for composite action in the negative moment region.

Table 17.5-3

Longitudinal Reinforcing Steel For Deck Slabs on Girders
for New Bridges and Deck Replacements, HL-93 Loading, Slab Thickness "T" ≥ 8"

Longitudinal Reinforcing Steel for Deck Slabs on Girders for Deck Replacements, HL-93 Loading, Slab Thickness “T” < 8”			
Slab Thickness “T” (Inches)	Girder Spacing “S”	Bar Size and Spacing (Inches)	
		Prestressed Girder Bridges	Steel Girder Bridges
		#4’s Top and Bottom, Continuity Reinforcement To Be Designed (Top)	#4’s Top and Bottom, Continuity Reinforcement** #6’s (Top)
6.5	4’-0”	7.0	7.0
6.5	4’-3”	7.0	7.0
6.5	4’-6”	7.0	7.0
6.5	4’-9”	7.0	7.0
6.5	5’-0”	7.0	7.0
6.5	5’-3”	7.0	7.0
6.5	5’-6”	7.0	7.0
6.5	5’-9”	6.5	6.5
6.5	6’-0”	6.5	6.5
6.5	6’-3”	6.5	6.5
6.5	6’-6”	6.5	6.5
6.5	6’-9”	6.0	6.0
6.5	7’-0”	6.0	6.0
7	4’-0”	8.0	8.0
7	4’-3”	8.0	8.0
7	4’-6”	8.0	8.0
7	4’-9”	8.0	8.0
7	5’-0”	8.0	8.0
7	5’-3”	8.0	8.0
7	5’-6”	8.0	8.0
7	5’-9”	7.5	7.5
7	6’-0”	7.5	7.5
7	6’-3”	7.5	7.5
7	6’-6”	7.0	7.0
7	6’-9”	7.0	7.0
7	7’-0”	7.0	7.0
7	7’-3”	6.5	6.5
7	7’-6”	6.5	6.5
7	7’-9”	6.5	6.5
7	8’-0”	6.0	6.0
7.5	4’-0”	9.0	9.0
7.5	4’-3”	9.0	9.0
7.5	4’-6”	9.0	9.0
7.5	4’-9”	9.0	9.0

7.5	5'-0"	9.0	9.0
7.5	5'-3"	9.0	9.0
7.5	5'-6"	9.0	9.0
7.5	5'-9"	8.5	8.5
7.5	6'-0"	8.5	8.5
7.5	6'-3"	8.5	8.5
7.5	6'-6"	8.0	8.0
7.5	6'-9"	8.0	8.0
7.5	7'-0"	7.5	7.5
7.5	7'-3"	7.5	7.5
7.5	7'-6"	7.5	7.5
7.5	7'-9"	7.0	7.0
7.5	8'-0"	7.0	7.0
7.5	8'-3"	6.5	6.5
7.5	8'-6"	6.5	6.5
7.5	8'-9"	6.5	6.5
7.5	9'-0"	6.0	6.0
7.5	9'-3"	6.0	6.0
7.5	9'-6"	5.5	5.5

Legend:

** Use for deck slabs on steel girders in negative moment regions when not designed for negative moment composite action.

Table 17.5-4

Longitudinal Reinforcing Steel for Deck Slabs
on Girders for Deck Replacements, HL-93 Loading, Slab Thickness "T" < 8"
(Only use Table 17.5-4 if Bridge Rating is unacceptable with "T" ≥ 8")

The longitudinal reinforcing steel presented in Table 17.5-3 and Table 17.5-4 is designed in accordance with *AASHTO LRFD*. The tables are developed based on deck concrete with a 28-day compressive strength of f'_c = 4 ksi and reinforcing steel with a yield strength of f_y = 60 ksi. The dead load includes 20 psf for future wearing surface.

The reinforcing bars presented in the "Bar Size and Spacing" column (the third column) in Table 17.5-3 and Table 17.5-4 are for one layer only. Identical steel should be placed in both the top and bottom layers, except for continuity steel.

17.5.3.3 Empirical Design of Slab on Girders

WisDOT policy item:

Approval from the Bureau of Structures Chief Structural Design Engineer is required for use of the empirical design method.

In addition to the traditional design method for decks, as described above, AASHTO also provides specifications for an empirical design method. This method, which is new to *AASHTO LRFD*, does not require the computation of design moments and is simpler to apply than the traditional design method. However, it is applicable only under specified design conditions. The empirical design method should not be used on bridge decks with heavy truck traffic. The empirical design method is described in **LRFD [9.7.2]**.

17.6 Cantilever Slab Design

For deck slabs on girders, the deck overhang must also be designed. Design of the deck overhang involves the following two steps:

1. Design for flexure in deck overhang based on strength and extreme event limit states.

2. Check for cracking in overhang based on service limit state.

The locations of the design sections are illustrated in Figure 17.6-1.

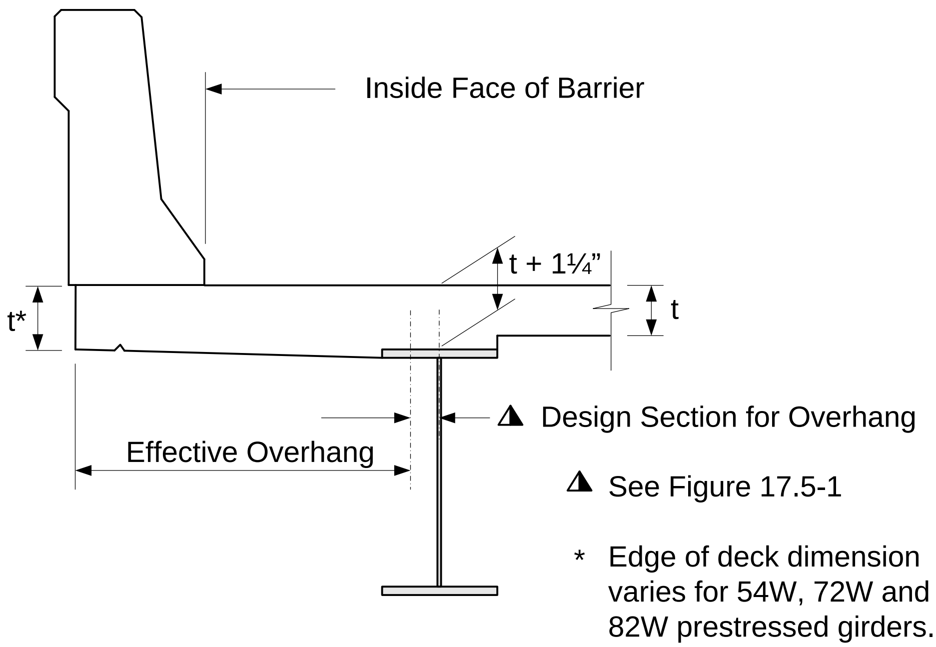


Figure 17.6-1
Deck Overhang Design Section

As described in **LRFD [A13.4]**, deck overhangs must be designed to satisfy three different design cases. These three design cases are summarized in Table 17.6-1.

Design Case	Applied Loads	Limit State	Design Locations
Design Case 1	Horizontal vehicular collision force and dead loads	Extreme Event II	At inside face of barrier At design section for overhang
Design Case 2 (usually does not control)	Vertical vehicular collision force and dead loads	Extreme Event II	At inside face of barrier At design section for overhang
Design Case 3	Dead and vehicle live loads	Strength I	At design section for overhang

Table 17.6-1
Deck Overhang Design Cases

The design load for Design Case 1 is a horizontal vehicular collision force, as illustrated in Figure 17.6-2. The transverse vehicle impact force, F_t, is specified in **LRFD [Table A13.2-1]** for various railing test levels. The force values specified in **LRFD [Table A13.2-1]** represent the total force, and neither dynamic load allowance nor multiple presence factors should be applied to these values.

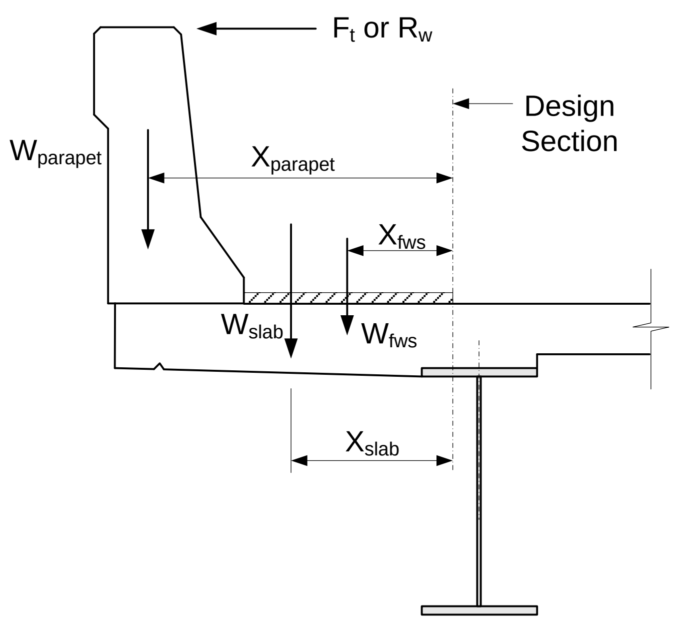


Figure 17.6-2
Design Case 1

The concrete barrier resistance, R_w, and the critical length of wall failure, L_c, are calculated in accordance with **LRFD [A13.3.1]**.

The longitudinal distribution length of the collision force for a continuous concrete barrier is calculated as illustrated in Figure 17.6-3. An angle of 30º is conservatively assumed for the load distribution from the front face of the barrier to the overhang design section.

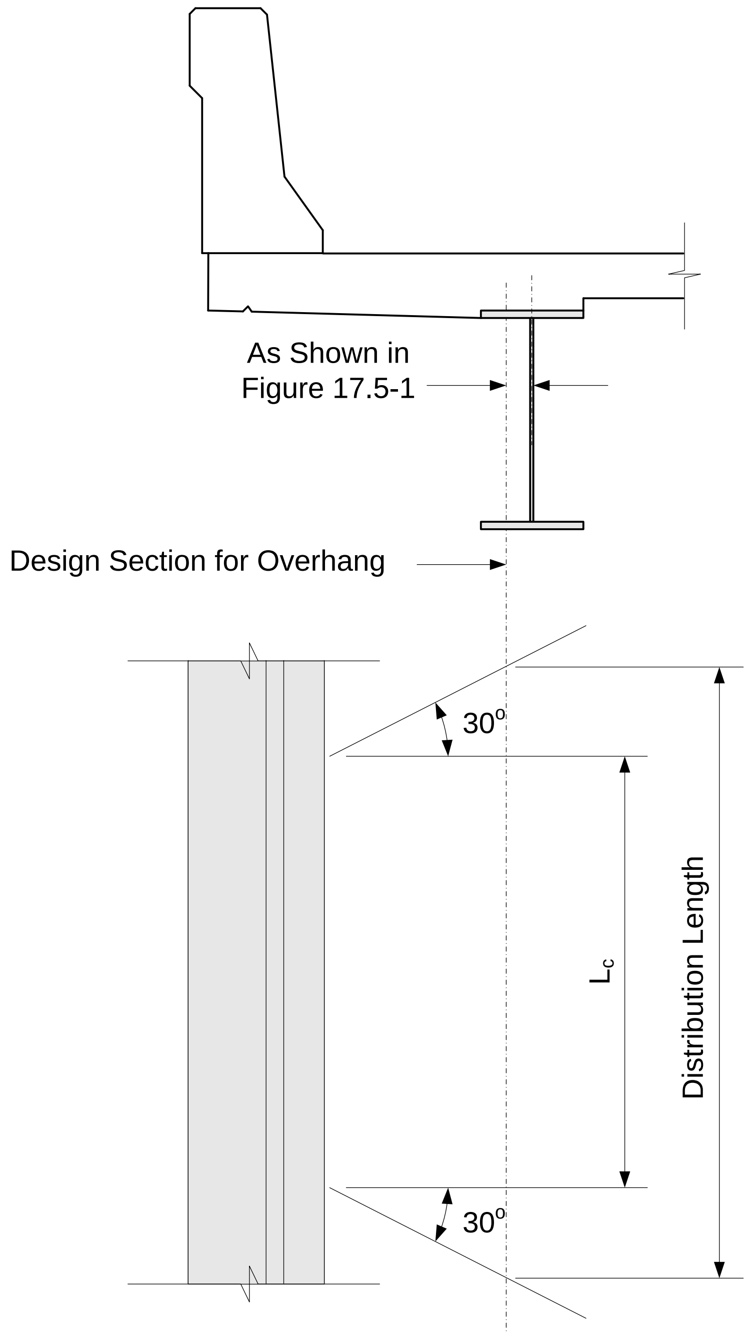


Figure 17.6-3
Assumed Distribution of Collision Moment Load in the Overhang

The design load for Design Case 2 is a vertical vehicular collision force, as illustrated in Figure 17.6-4. The vertical design force, F_v, is specified in **LRFD [Table A13.2-1]** for various railing test levels. The values for F_v specified in **LRFD [Table A13.2-1]** represent the total force, and neither dynamic load allowance nor multiple presence factors should be applied to these values.

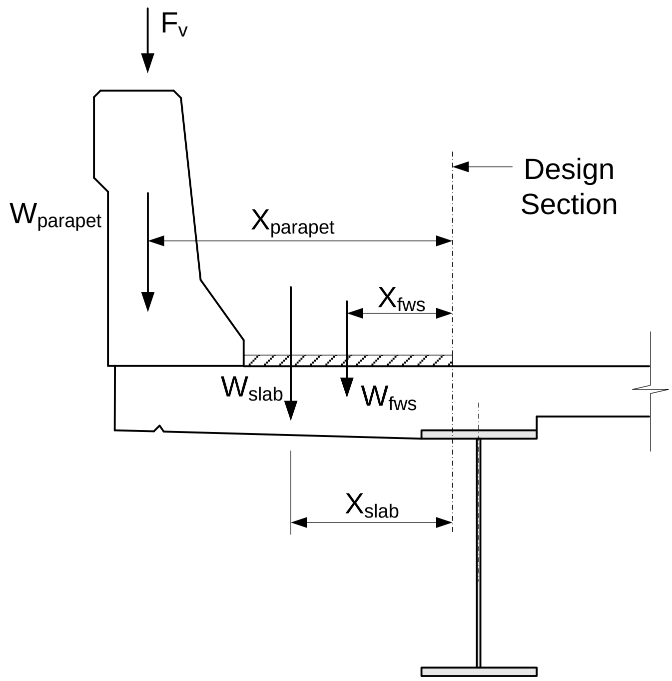


Figure 17.6-4
Design Case 2

For continuous concrete barriers, Design Case 2 generally does not control.

For steel post and beam railing, the overhang design is as specified in **LRFD [A13.4.3.1]**, and the assumed effective length of the cantilever for carrying concentrated post loads (either transverse or vertical) is as shown in Figure 17.6-5.

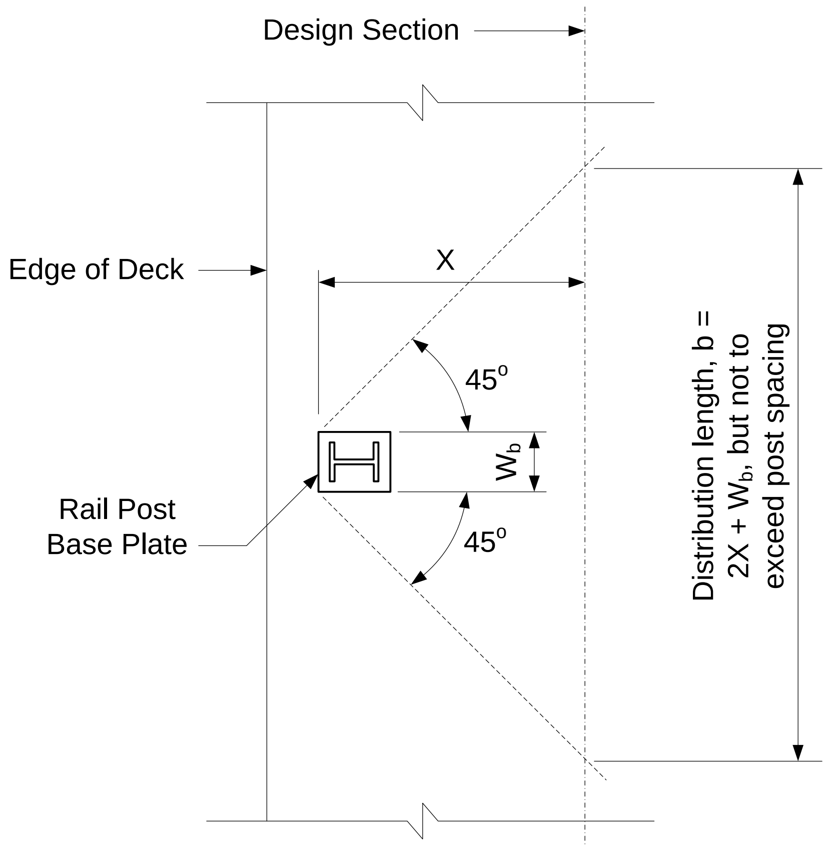


Figure 17.6-5
Effective Length of Cantilever for Carrying Concentrated Post Loads

As used in Figure 17.6-5:

b	=	Effective length of cantilever for carrying concentrated post loads (inches)
W_b	=	Width of base plate (inches)
X	=	Distance from edge of base plate nearest to edge of deck to design section (inches)

For steel post and beam railing, the punching shear force is computed as specified in **LRFD [A13.4.3.2]**, and the assumed distribution of forces for punching shear is as shown in Figure 17.6-6.

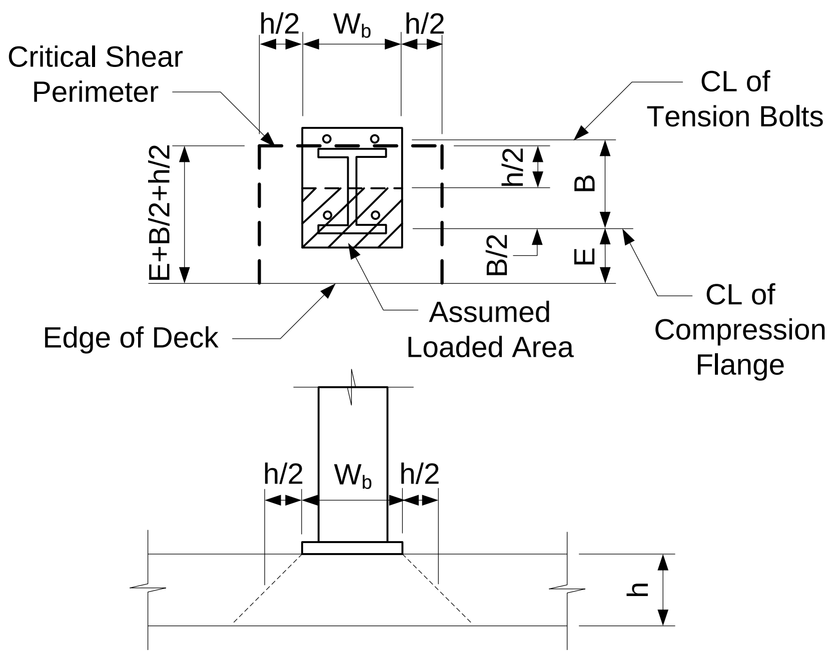


Figure 17.6-6
Assumed Load Distribution for Punching Shear

As used in Figure 17.6-6:

B = Distance between centroids of tensile and compressive stress resultants in post (inches)

E = Distance from edge of slab to centroid of compressive stress resultant in post (inches)

h = Depth of slab (inches)

W_b = Width of base plate (inches)

The design loads for Design Case 3 are dead and live loads, as illustrated in Figure 17.6-7.

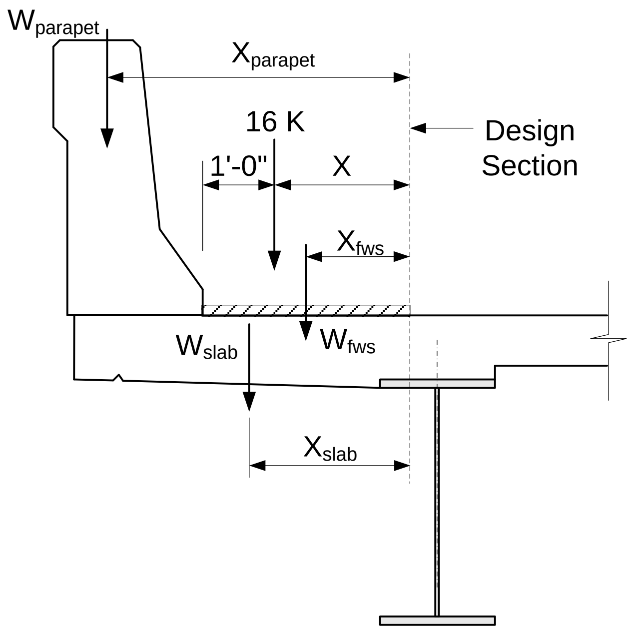


Figure 17.6-7
Design Case 3

As presented in **LRFD [Table 4.6.2.1.3-1]**, the equivalent strip (in the longitudinal direction), in units of inches, for live load on an overhang for Design Case 3 is:

Equivalent strip = 45.0 + 10.0X

Where:

X = Distance from load to point of support (feet), as illustrated in Figure 17.6-7

The multiple presence factor of 1.20 for one lane loaded and a dynamic load allowance of 33% should be applied, and the moment due to live load and dynamic load allowance is then computed.

Based on the computations for the three design cases, the controlling design case and design location are identified. The factored design moment is used to compute the required reinforcing steel. Cracking in the overhang must be checked for the service limit state in accordance with **LRFD [5.7.3.4]**. The controlling overhang reinforcement for cantilever deck slabs is shown in Table 17.6-2 and Table 17.6-3 for single slope and sloped face concrete parapets, and in Table 17.6-4 and Table 17.6-5 for steel railing Type “NY”/“M”. Type “W” railing is no longer allowed on girder structures.

If the area of reinforcing steel required in the overhang is greater than the area of reinforcing steel provided by the standard transverse reinforcement over the interior girders, it shall be placed as detailed in Figure 17.6-8.

17.6.1 Rail Loading for Slab Structures

For concrete slab superstructures, the designer is required to consider the rail loading and provide adequate transverse reinforcing steel, accordingly. The top transverse slab reinforcement for both concrete parapet and steel railing Type “NY”, "M" or "W" are shown on the Standard Details.

17.6.2 WisDOT Overhang Design Practices

<u>WisDOT policy item:</u>

Current design practice in Wisconsin limits the standard slab overhang length to 3’-7”, measured from the centerline of the exterior girder to the edge of the slab. A 4’-0” overhang is allowed for some wide flange prestressed concrete girders (54W”, 72W”, 82W”). A 4’-6” overhang may be used where a curved roadway is placed on straight girders at the discretion of the designer. The total overhang when a cantilevered sidewalk is used is limited to 5’-0”, measured from the centerline of the exterior girder to the edge of the sidewalk. A minimum of 6” from the edge of the top flange to the edge of the deck should be provided, with 9” preferred.

The overhang length has been limited to prevent rotation of the girder and bending of the girder web during construction caused by the eccentric load from the cantilevered forming brackets. The upper portion of these brackets attaches to the girder top flange, and the lower portion bears against the girder web. If the girder rotates or the web bends at the bracket bearing point, the end of the bracket will move downward because of bracket rotation. If the rails supporting the paving machine are located near the end of the bracket, the paving machine will move downward more than the girder and the anticipated profile grade line will not be achieved. Factors affecting girder rotation are diaphragm spacing, stiffness, connections and girder torsional stiffness. Factors affecting web bending are stiffener spacing and web thickness. Do not place a note or detail on the plan for exterior girder bracing required by the contractor as this is covered by the specs.

In the following tables, the slab thickness, "t", is the slab thickness between interior girders. The area of steel shown in the following tables is the controlling value from Design Case 1, 2 or 3. The value shown is the larger area of steel required at the front face of the barrier or at the design section. Girder Type 1 shall include steel girders and the following prestressed girders; 28-inch, 36-inch, 36W-inch, 45W-inch. Prestressed girders used for rehabilitation projects, 45-inch, 54-inch and 70-inch, shall also be considered to be Girder Type 1. Girder Type 2 shall include the following prestressed girders; 54W-inch, 72W-inch and 82W-inch.

If the area of reinforcing steel required in the overhang is greater than the area of reinforcing steel provided by the standard transverse reinforcement over the interior girders, reinforcement must be added to satisfy the overhang design requirements. The amount of reinforcement that must be added in the overhang is the amount required to satisfy the overhang design requirement minus the amount provided by the standard transverse reinforcement over the interior girders. This additional reinforcement should be carried for the bar development length past the exterior girder centerline. The reinforcement shall be placed as detailed in Figure 17.6-8. Use either a number 4 or 5 bar to satisfy this requirement. The additional bar shall be placed at one or two times the standard transverse bar spacing as required.

Effective Overhang (Feet)	Deck Thickness Between Girders, “t” (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.75	0.749	0.690	0.640	0.597	0.562	0.529	0.514
2.00	0.747	0.690	0.643	0.603	0.568	0.536	0.510
2.25	0.766	0.706	0.655	0.612	0.576	0.545	0.517
2.50	0.781	0.718	0.666	0.622	0.584	0.551	0.523
2.75	0.793	0.728	0.675	0.629	0.591	0.557	0.527
3.00	0.805	0.738	0.682	0.636	0.596	0.562	0.532
3.25	0.815	0.745	0.688	0.642	0.601	0.566	0.535
3.50	0.824	0.752	0.694	0.646	0.605	0.569	0.538
3.75	0.849	0.761	0.700	0.650	0.608	0.572	0.541
4.00	0.959	0.862	0.785	0.688	0.636	0.590	0.544

Table 17.6-2

Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Single Slope or Sloped Face Concrete Parapets --- Girder Type 1

Effective Overhang (Feet)	Deck Thickness Between Girders, “t” (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.25	0.749	0.691	0.644	0.603	0.568	0.537	0.511
1.5	0.761	0.700	0.649	0.607	0.570	0.537	0.510
1.75	0.761	0.700	0.649	0.606	0.570	0.537	0.510
2	0.761	0.700	0.649	0.606	0.570	0.537	0.510
2.25	0.740	0.681	0.632	0.591	0.555	0.547	0.526
2.5	0.735	0.678	0.629	0.588	0.553	0.559	0.541
2.75	0.732	0.674	0.626	0.586	0.550	0.549	0.557
3	0.730	0.673	0.626	0.584	0.550	0.539	0.553
3.25	0.729	0.672	0.624	0.584	0.549	0.528	0.543

Table 17.6-3

Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Single Slope or Sloped Face Concrete Parapets --- Girder Type 2

Effective Overhang (Feet)	Deck Thickness Between Girders, “t” (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.75	0.277	0.277	0.277	0.277	0.251	0.202	0.159
2.00	0.287	0.287	0.287	0.287	0.264	0.220	0.180
2.25	0.295	0.295	0.295	0.295	0.274	0.234	0.198
2.50	0.302	0.302	0.302	0.302	0.282	0.246	0.212
2.75	0.307	0.307	0.307	0.307	0.290	0.255	0.224
3.00	0.312	0.312	0.312	0.312	0.295	0.278	0.263
3.25	0.394	0.394	0.394	0.394	0.392	0.389	0.340
3.50	0.465	0.465	0.465	0.465	0.464	0.436	0.412
3.75	0.497	0.497	0.497	0.497	0.477	0.489	0.480
4.00	0.567	0.567	0.567	0.567	0.542	0.501	0.504

<u>Table 17.6-4</u>

Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Tubular Railing Type “NY”/”M”
Girder Type 1

Effective Overhang (Feet)	Deck Thickness Between Girders, “t” (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.25	0.542	0.435	0.345	0.272	0.213	0.161	0.117
1.5	0.542	0.435	0.345	0.272	0.213	0.161	0.117
1.75	0.525	0.435	0.345	0.272	0.213	0.161	0.117
2	0.423	0.423	0.345	0.269	0.203	0.147	0.096
2.25	0.290	0.280	0.228	0.185	0.146	0.114	0.128
2.5	0.237	0.237	0.217	0.176	0.151	0.146	0.160
2.75	0.275	0.275	0.275	0.263	0.247	0.234	0.222
3	0.269	0.269	0.269	0.269	0.269	0.256	0.244
3.25	0.334	0.334	0.334	0.334	0.334	0.330	0.314

<u>Table 17.6-5</u>

Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Tubular Railing Type “NY”/“M”
Girder Type 2

Notes:

1. Tables show the total area of transverse deck reinforcement required per foot.

2. The values in Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 are based on the following design criteria:

   - f'c = 4 ksi
   - fy = 60 ksi
   - Top steel clearance = 2 1/2"
   - Effective Overhang as illustrated in Figure 17.6-1

3. For Tubular Railing Type "NY"/"M", the No. 6 "U" bars located at the rail post locations should not be included when calculating the total available area of reinforcement.

4. The values in the shaded region are satisfied by the standard transverse reinforcement for all girder spacings and standard transverse deck reinforcement. No additional checks or reinforcement are required.

5. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail "A" shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail "B" shall be used with Table 17.6-4.

6. For bridge decks with raised sidewalks, the additional reinforcement shown in Table 17.6-2, Table 17.6-3, Table 17.6-4, and Table 17.6-5, need not be used. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for information pertaining to the additional reinforcement to be used at raised sidewalks.

Example Use of Tables:

Given Information:

54W" PSG, 15" from CL girder to Design Section -- (Girder Type 2)

Girder Spacing = 7'-0"

Overhang = 3'-0", Effective Overhang = 1'-9"

Type "NY" rail

From Table 17.5-1:

Deck thickness = 8"

Design Section at 15", use #5's @ 8.5", As provided = 0.43 in^2/ft

From Table 17.6-5:

Transverse area of steel required = 0.542 in^2/ft

Therefore:

Additional area of steel required = 0.542 - 0.43 = 0.112 in^2/ft

Use either one or two times the spacing of the standard transverse reinforcement.

Lapping every other bar: use #4's @ 17", As = 0.14 in^2/ft, use Detail "A".

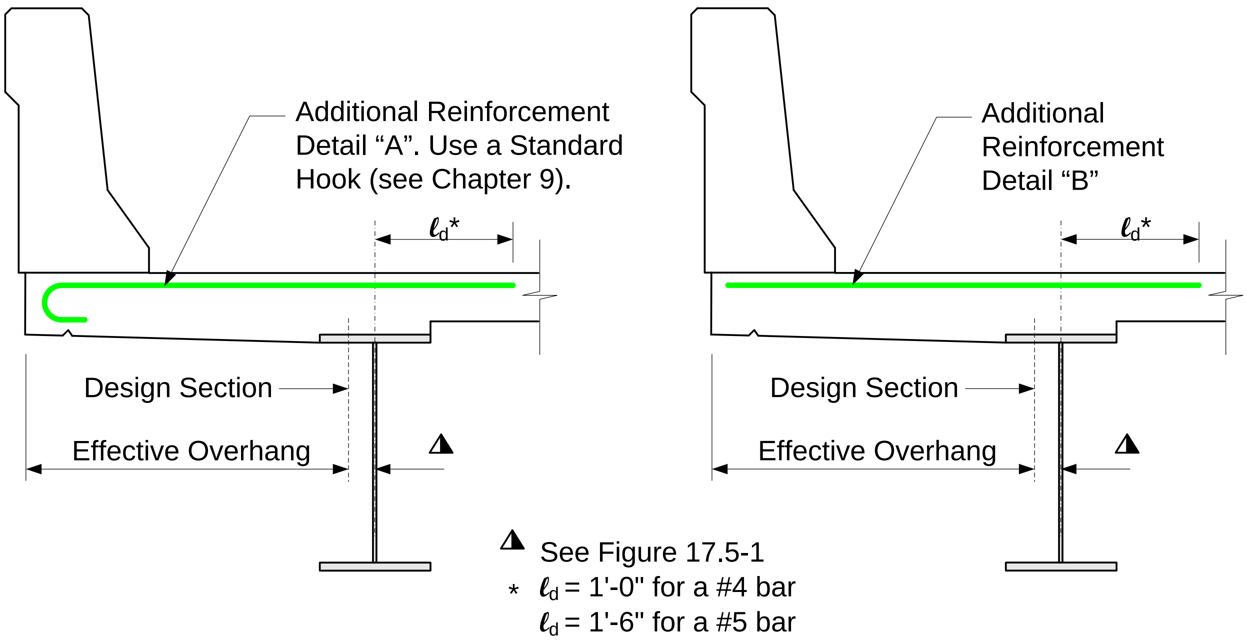


Figure 17.6-8
Overhang Reinforcement Details

To reiterate:

1. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail "A" shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail "B" shall be used with Table 17.6-4.

2. Girder Type 1 shall include steel girders and the following prestressed girders; 28-inch, 36-inch, 36W-inch, 45W-inch. Prestressed girders used for rehabilitation projects, 45-inch, 54-inch and 70-inch, shall also be considered to be Girder Type 1. Girder Type 2 shall include the following prestressed girders; 54W-inch, 72W-inch and 82W-inch.

17.7 Construction Joints

Optional transverse construction joints are permitted on continuous concrete deck structures to limit the concrete volume in a single pour. Refer to the Standard Detail for Slab Pouring Sequence for the optimum slab pouring sequence. On steel structures over 300 feet long, transverse construction joints, if used, are to be placed at 0.6 of the span length beyond the pier in the direction of the pour. For continuous prestressed concrete girder bridges, optional transverse construction joints should be located midway between the cut-off points for continuity reinforcing steel or at 0.75 of the span, whichever is closest to the pier.

The rate of placing concrete for continuous steel girders shall equal or exceed 0.5 of the span length per hour but need not exceed 100 cubic yards per hour. Transverse construction joints may be omitted with approval of Bureau of Structures.

When the deck width of a girder superstructure exceeds 90 feet or the width of a slab superstructure exceeds 52 feet, a longitudinal construction joint with reinforcement through the joint shall be detailed. Longitudinal joints should not be located directly above girders and should be at least 6 inches from the edge of the top flange of the girder. Longitudinal joints are preferably located beneath the median or parapet. Otherwise, the joint should be located along the edge of the lane line or in the middle of the lane. The longitudinal construction joint should be used for staged construction and for other cold joint applications within the deck. A longitudinal construction joint detail is provided in Standard Detail 17.02 – Deck and Slab Details.

Optional longitudinal construction joints shall be detailed accordingly in the plans. Longitudinal construction joints requested by the contractor are to be approved by the engineer. Optional and contractor requested joints are to be located as previously mentioned.

Open joints may be used in a median or between parapets. Considerations should be given to sealing open joints with compression seals or other sealants.

The structure plans should permit the contractor to propose an alternate construction joint schedule subject to approval of the engineer.

17.8 Bridge Deck Protective Systems

17.8.1 General

FHWA encourages states that require the use of de-icers to employ bridge deck protective systems. The major problem resulting in bridge deck deterioration is delamination of the concrete near the top mat of the reinforcing steel followed by subsequent spalling of the surface concrete. Research shows that the most prevalent cause of extensive deck deterioration is corrosion of the reinforcing steel due to the intrusion of chlorides into the concrete from repeated de-icer applications during snow and/or ice removal.

Several types of bridge deck protective systems are currently available. Some have been approved by FHWA based on their initial performance. Some of the more common types of protective systems are epoxy coated reinforcing steel, galvanized or stainless steel reinforcing steel, microsilica modified concrete or polymer impregnated concrete, cathodic protection and deck sealers. Epoxy coated reinforcing steel and deck sealers preferred by WisDOT.

Structures other than box culverts that are designed to carry an earth fill are required to have waterproofing membrane systems on the deck to protect the slab. This includes bridges designed for future grade changes.

17.8.2 Design Guidance

All deck reinforcement bars shall be epoxy coated and the top reinforcing bars shall have a minimum of 2 ½ inches of cover.

All decks shall receive a protective surface treatment. Other locations for protective surface treatment should include: parapet, parapet wing, median, sidewalk and edge of deck/slab and 1’-0” underside of deck/slab when open railings are utilized.

Additional protective systems may be desired to minimize future rehabilitations. One or a combination of systems may be used on large projects such as Mega Projects. Contact the WisDOT Bureau of Structures Design Section for approval and project specific guidance. The following systems are currently being used and should be considered on new structures and deck rehabilitations:

- High Performance Concrete (HPC) – This is typically used within the bridge superstructure (deck, diaphragms, parapets, structural approach slabs, etc.) on urban interchange projects

- Polymer overlays - This system extends the decks service life before rehabilitation is required.

- Stainless steel deck reinforcement – Use of stainless steel in lieu of epoxy bars may be justified for urban interchange projects and complex structures. Savings from reducing the number of rehabilitation projects and user costs can be substantial. Currently, only the enhanced corrosion protection benefits shall be utilized and reinforcement shall be selected per the epoxy coated deck design tables. The use of stainless reinforcing steel shall be approved by Chief Structures Development or Design Engineer and may require a life cycle analysis.

17.9 Bridge Approaches

The structure approach slab, or approach pavement, is part of the roadway design plans. Structure approach standards are provided in the *Facilities Development Manual* (*FDM*).

Guidance for the selection of pavement types for bridge approaches is as shown in *FDM* Procedure 14-10-15.

Considerations for site materials, drainage and backfill are provided in Chapter 12 – Abutments. Most approach pavement failures are related to settlement of embankment or foundation materials. Past experience shows that significant settlement is most likely to occur where marginal materials are used. Designers are encouraged to provide perforated underdrains wrapped in geotextile fabric placed in a trench filled with crushed stone. Also, abutment backfill material should be granular in nature and consolidated under optimum moisture conditions.

17.10 Design of Precast Prestressed Concrete Deck Panels

17.10.1 General

An advantage of stay-in-place forms is that they can be placed in less time than it takes to place the forms for a conventional deck. There is also a labor savings because the extra step of removing deck forms is not required. Stay-in-place forms are often the preferred system for shallow box girders because of the difficulty of removing forms in a confined space.

If not detailed in the contract documents, precast concrete deck panels may be used at the option of the contractor, provided the specifications permit their use. A standardized special provision (STSP) for optional use of precast prestressed concrete deck plans is available from the Bureau of Highway Construction, Standards Development Section.

When a contractor elects to use precast deck panels at their option, the contractor is responsible for the shop plans of the panels and any other changes that may be required to the reinforcing steel in the cast-in-place portion of the deck. Payment to a contractor who chooses to use stay-in-place forms is based on the contract prices bid for the conventional cast-in-place deck.

Deck panels are only used between the inside faces of the exterior girders. The overhangs outside the exterior girders are formed and the concrete placed in the same way as in a conventional cast-in-place deck. On skewed decks, the contractor may form and cast the skewed portion of the deck full depth or they may use skewed end deck panels which may be individually precast or saw-cut from square end planks.

A problem with decks formed with concrete deck panels is that cracks often form in the cast-in-place concrete over the transverse joints between panels and along the edges of the panels parallel to the girders. Reflection cracking is less of a problem when these panels are used on prestressed concrete girders than on steel girders. Simple-span prestressed concrete girder bridges have less reflective cracking than continuous-span prestressed concrete girder bridges.

17.10.2 Deck Panel Design

The design of precast prestressed concrete deck panels shown in Table 17.10-1 is based on *AASHTO LRFD* design criteria. These panels were designed for flexure due to the HL-93 design truck live load, dead load of the plastic concrete supported by the panels, a construction load of 50 psf, dead load of the panels and a future wearing surface of 20 psf. The live load moments were obtained from **LRFD [Table A4-1]**.

At the request of precast deck panel fabricators, only two strand sizes are used – 3/8 inch and 1/2 inch. Precast deck panel fabricators do not want the additional overhead expense of stocking 7/16-inch strand. Strand spacing is given in multiples of 2 inches.

WisDOT exception to AASHTO:

A 3-inch minimum panel thickness is used, even though **LRFD [9.7.4.3.1]** specifies a minimum thickness of 3.5 inches.

The decision to use a 3-inch minimum panel was based on the successful use of 3-inch panels by other agencies over many years. In addition, a minimum of 5 inches of cast-in-place concrete is preferred for crack control and reinforcing steel placement. A 3.5-inch panel thickness would require an 8.5-inch deck, which would not allow direct substitution of panels for a traditionally designed 8-inch deck.

A study performed at Iowa State University determined that a 3-inch thick panel with coated 3/8-inch strands at midthickness spaced at 6 inches, along with epoxy-coated 6 x 6 – D6 x D6 welded wire fabric, was adequate to prevent concrete splitting during strand detensioning. The use of #3 bars placed perpendicular to the strands at 9" spacing also prevents concrete splitting.

Panel thicknesses were increased by ½ inch whenever a strand spacing of less than 6 inches was required. Strands with a ½-inch diameter were used in panels 3½ inches thick or greater when 3/8-inch strands spaced at 6 inches were not sufficient.

The allowable tensile stress in the panels, as presented in **LRFD [Table 5.9.4.2.2-1]**, is as follows:

$0.0948\lambda\sqrt{f'_c} \leq$ 0.3 ksi ; where λ = conc. density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight conc.

This allowable tensile stress limit is based on f'_c in units of ksi and is for components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions.

The transfer length of the strands is assumed to be 60 strand diameters at a stress of 202.5 ksi. The development length, L_d, of the strands, as presented in **LRFD [5.11.4.2]**, is assumed to be as follows:

$$L_d = k\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

k	=	1.0 for pretensioned members with a depth less than 24 inches
d_b	=	Nominal strand diameter (inches)
f_{ps}	=	Average stress in prestressing steel at the time when the nominal resistance of the member is required (ksi)

f_{pe} = Effective stress in prestressing steel after losses (ksi)

L_d = Development length beyond critical section (inches)

The minimum panel width is the length required for the panel to extend 4" onto the top flange as shown in Table 17.10-1. A linear reduction in f_{pe} is required if the panel width is less than two times the development length. The values shown in Table 17.10-1 consider this linear reduction.

The designs in Table 17.10-1 are based on uncoated prestressing strands. Grit-impregnated, epoxy-coated strands cost four times as much as uncoated strands but require about half the transfer and development length as uncoated strands. A cover of 1 1/4 inches is adequate to provide protection from chlorides for uncoated strands using a 5 ksi concrete mix. However, for bridges with high traffic volume, a 6 ksi mix is recommended.

LRFD [9.7.4.3.2] specifies that the strands need not extend beyond the panels into the cast-in-place concrete above the beams. This simplifies construction of the panels at the plant since they can be saw cut to the required length. Installation in the field is also simplified because extended strands may interfere with girder shear connectors. As a substitute for the strands that don't extend out of the panels, #4 bars spaced at twice the spacing of the transverse bars are placed on top of the panels over the girders in the cast-in-place concrete. These bars anchor the panels together to prevent or reduce longitudinal cracking over the ends of the panels and also resist any positive continuity moments that may develop. Also by not extending the strands into the cast-in-place concrete, the uncoated strands are not exposed to chlorides that may seep through cracks that may develop in the cast-in-place concrete.

LRFD [5.7.3.3.2] requires that the moment capacity of a flexural member be greater than the cracking moment based on the modulus of rupture. This requirement may be waived if the moment capacity is greater than 1.33 times the factored design moment. The purpose of this requirement is to provide a minimum amount of reinforcement in a flexural member so that a flexural failure will not be sudden or occur without warning. Tests have shown that for slabs on girders, the failure mode is a punching shear failure and not a flexural failure. ACI 10.5.4 also recognizes the difference between slabs and beams and does not require the same minimum reinforcement for slabs. For these reasons, **LRFD [5.7.3.3.2]** was not considered in the designs of the panels shown in Table 17.10-1. However, panels with a width of 6 feet or more meet the requirements of **LRFD [5.7.3.3.2]**.

17.10.3 Transverse Reinforcement for Cast-in-Place Concrete on Deck Panels

The design of the transverse reinforcing steel in the cast-in-place concrete placed on deck panels is based on *AASHTO LRFD*. The live load moments used to determine the size and spacing of the transverse reinforcing bars placed in the top of the cast-in-place concrete are from **LRFD [Table A4-1]**. The reinforcing steel in the cast-in-place concrete is also designed for a future wearing surface of 20 psf. With stay-in-place forms, there are no negative moments from the dead load of the cast-in-place concrete. The required reinforcing steel shown in Table 17.10-2 is based on both the strength requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.7.3.4]** and as shown in 17.5.3.1. A concrete strength of 4 ksi was assumed, and the haunch height over the girders was not considered.

The distance from the centerline of the girder to the design section is from **LRFD [4.6.2.1.6]**. For prestressed concrete girders, use the values in Figure 17.5-1.

The reinforcing steel in Table 17.10-2 does not account for deck overhangs. However, Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 provide the minimum reinforcing steel required in the overhangs. Also for any portion of a deck not supported by deck panels, use Table 17.5-1 for determining the required reinforcing steel.

17.10.3.1 Longitudinal Reinforcement

For continuous prestressed concrete girders, the longitudinal reinforcing steel over the piers is the same as that required for a conventional deck. For steel girders, see 17.5.3.2 for longitudinal continuity reinforcement.

17.10.4 Details

Precast deck panels should extend a minimum of 1.5 inches beyond the face of concrete diaphragms at the substructure units. The transverse joints between panels in adjacent bays should be staggered, preferably a distance about 1/2 panel length. Staggering the joints helps to minimize transverse reflective cracking.

Panels should never rest directly on a girder flange. According to **LRFD [9.7.4.3.4]**, "The ends of the formwork panels shall be supported on a continuous mortar bed or shall be supported during construction in such a manner that the cast-in-place concrete flows into the space between the panel and the supporting component to form a concrete bedding." The minimum width of bearing on the flange of a girder for both concrete and mortar or grout support is 3 inches. See Figure 17.10-1 and Figure 17.10-2 for additional information.

High-density expanded polystyrene is used to support the panels prior to the placement of the cast-in-place concrete under the panel. The polystyrene is cut to the required haunch height so a constant slab thickness is maintained. High-density expanded polystyrene is available in different strengths, and it is the responsibility of the contractor to determine the strength required based on the vertical load that must be resisted. Fiber board or sheathing panel supports are not allowed because the slight deflection of polystyrene compresses the concrete underneath the panel and results in less reflective longitudinal cracking along the panel edge.

When panels are supported on grout, the main function of the polystyrene is to form the haunch height and to form a dam for the grout placement. The grout must be placed immediately before placement of the panels. It is important that enough grout be placed so that the vertical load from the panels is supported by the grout and not by the polystyrene.

Some agencies specify a maximum haunch height. When it is exceeded, they allow the contractor to thicken the slab. Wisconsin does not specify a maximum haunch height and leaves that decision to the designer, who is better informed to make that decision based on the specific situation of their project.

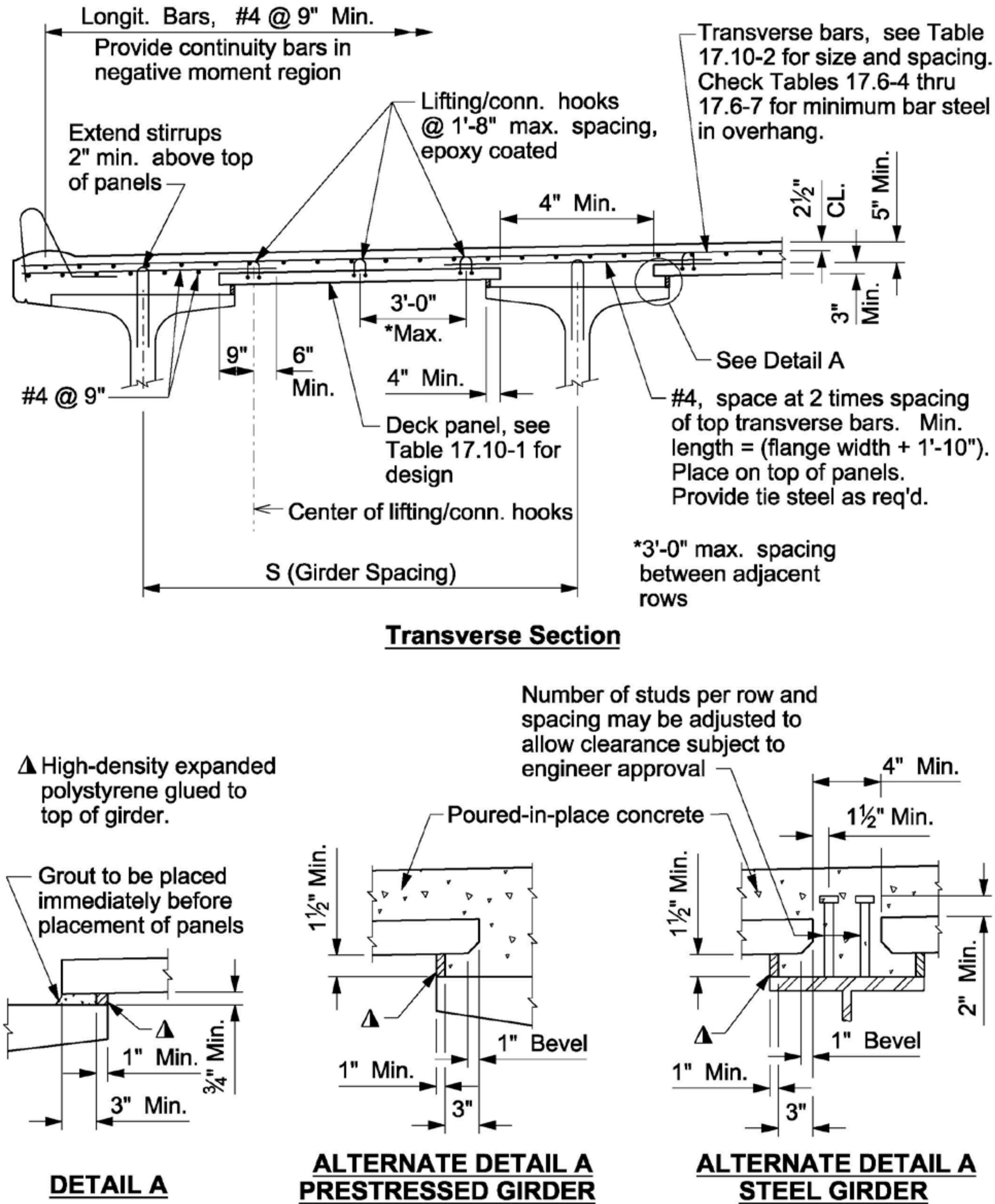
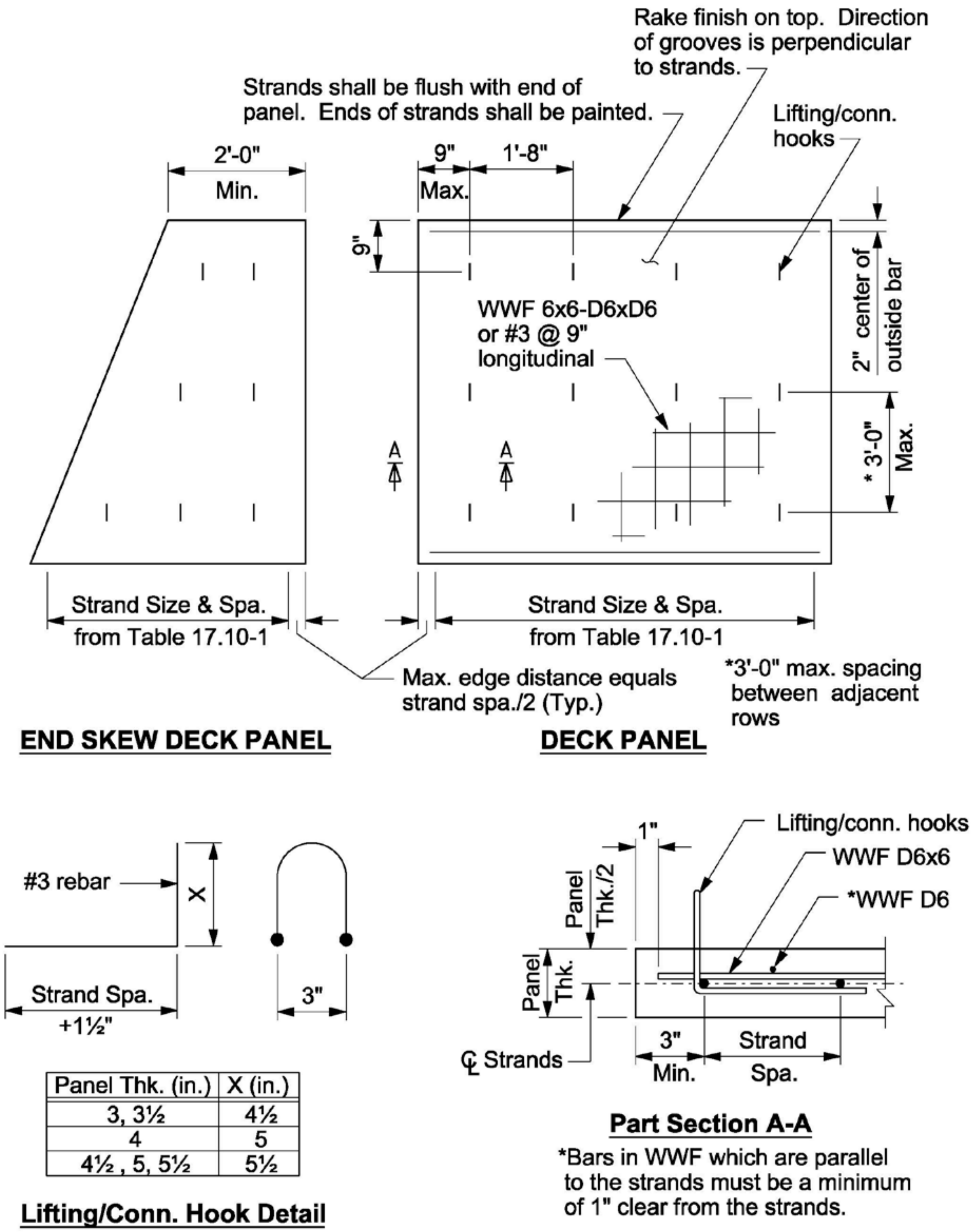


Figure 17.10-1
Transverse Section through Slab on Girders with Deck Panel and Details



Panel Thk. (in.)	X (in.)
3, 3½	4½
4	5
4½ , 5, 5½	5½

Figure 17.10-2
Deck Panel Details

Girder Spacing "S"	Panel Thick. (Inches)	Total Slab Thick. (Inches)	Top Flange Width (Inches)												
			12		16		18		24		30		30		
			Strand		Strand		Strand		Strand		Strand		Strand		
			Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	Spa. Inches	Pi Kips	
4'-6"	3	8	10	13.17	10	12.33	10	11.92	10	11.08	10	11.08	10	11.08	3/8" Diameter Strands
4'-9"	3	8	10	13.58	10	12.75	10	12.75	10	11.50	10	11.08	10	11.08	
5'-0"	3	8	10	14.42	10	13.58	10	13.17	10	12.33	10	11.08	10	11.08	
5'-3"	3	8	10	14.83	10	14.00	10	13.58	10	12.75	10	11.92	10	11.08	
5'-6"	3	8	10	15.67	10	14.83	10	14.42	10	13.17	10	12.33	10	11.08	
5'-9"	3	8	10	16.50	10	15.67	10	15.25	10	14.00	10	13.17	10	11.50	
6'-0"	3	8	8	14.25	10	16.50	10	16.08	10	14.83	10	13.58	10	11.50	
6'-3"	3	8	8	15.45	8	14.25	10	16.92	10	15.67	10	14.42	10	11.92	
6'-6"	3	8	8	16.12	8	15.45	8	14.78	10	16.50	10	15.25	10	12.33	
6'-9"	3	8	8	17.12	8	16.12	8	15.78	8	14.25	10	16.08	10	13.17	
7'-0"	3	8	6	14.19	8	17.12	8	16.45	8	15.45	8	14.25	10	13.58	
7'-3"	3	8	6	14.94	6	14.19	6	13.62	8	16.12	8	15.12	10	14.42	
7'-6"	3	8	6	15.69	6	14.94	6	14.69	8	17.12	8	15.78	10	15.25	
7'-9"	3	8	6	16.44	6	15.69	6	15.44	6	14.44	8	16.78	10	16.50	
8'-0"	3	8	6	17.19	6	16.44	6	16.19	6	15.19	6	14.19	8	14.25	
8'-3"	3.5	8.5	6	16.76	6	16.01	6	15.76	6	14.76	6	13.47	8	14.14	
8'-6"	3.5	8.5	10	29.48	6	16.76	6	16.51	6	15.51	6	14.51	8	14.97	
8'-9"	3.5	8.5	8	26.44	10	30.06	10	29.06	6	16.26	6	15.26	8	15.97	
9'-0"	3.5	8.5	8	27.44	8	26.44	8	26.10	6	17.01	6	16.01	8	16.64	
9'-3"	3.5	8.5	8	28.77	8	27.77	8	27.10	10	30.06	6	16.76	6	14.01	
9'-6"	4	9	8	27.76	8	26.76	8	25.95	10	29.22	6	16.37	8	17.20	
9'-9"	4	9	8	29.09	8	27.76	8	27.43	10	30.62	6	17.12	6	14.37	
10'-0"	4	9	8	30.09	8	29.09	8	28.43	8	27.09	10	30.20	6	15.12	
10'-3"	4	9	6	25.48	8	30.09	8	29.76	8	28.09	8	26.76	6	15.87	
10'-6"	4	9	6	26.23	6	25.48	8	30.76	8	29.09	8	27.76	6	16.62	
10'-9"	4	9.5	6	26.73	6	25.73	6	25.23	8	29.43	8	27.76	6	16.12	
11'-0"	4	9.5	6	27.48	6	26.73	6	26.23	8	30.43	8	28.76	6	16.87	
11'-3"	4	9.5	6	28.48	6	27.48	6	26.98	6	25.73	8	30.09	10	30.20	1/2" Diameter Strands
11'-6"	4	9.5	6	29.48	6	28.48	6	27.98	6	26.73	6	25.23	8	25.95	
11'-9"	4	10	6	30.23	6	28.98	6	28.48	6	26.98	6	25.48	8	25.95	
12'-0"	4.5	10	6	29.62	6	28.62	6	28.12	6	26.62	6	25.37	8	26.50	
12'-3"	4.5	10	6	30.62	6	29.62	6	29.12	6	27.62	6	26.12	8	27.83	
12'-6"	5	10	6	30.34	6	29.34	6	28.84	6	27.59	6	26.34	8	28.28	
12'-9"	5	10.5	6	30.59	6	29.59	6	29.09	6	27.59	6	26.34	8	27.95	
13'-0"	5.5	10.5	6	30.36	6	29.36	6	29.11	6	27.61	6	26.36	8	28.77	
13'-3"	5.5	10.5	4	23.52	6	30.36	6	29.86	6	28.61	6	27.36	8	29.77	
13'-6"	5.5	10.5	4	24.18	4	23.52	4	23.18	6	29.36	6	28.11	8	30.77	

13'-9"	6	11	4	23.39	6	30.41	6	30.16	6	28.66	6	27.41	8	29.96	
14'-0"	6	11	4	24.06	4	23.39	4	23.06	6	29.66	6	28.16	8	30.96	

Table 17.10-1

Precast Prestressed Concrete Deck Panel Design Table

Notes:

- Designed per AASHTO LRFD Specifications with HL 93 Loading.
- f'c = 6.0 ksi
- f'ci = 4.4 ksi
- f'c slab = 4.0 ksi
- f's = 270 ksi (low relaxation)
- Design loading includes 20 psf for future wearing surface and 50 psf for construction load. Pi's in Table are a minimum and may be increased to a maximum of 0.75 x f_s x A_s. Strands are located at the centroid of the panels.

Girder Spacing "S"	Total Slab Thick. Inches	Distance From C/L of Girder to Design Section (Inches)					
		3	4	5	6	10	15
4'-6"	8	#4 @ 9	#4 @ 9.5	#4 @ 10	#4 @ 10	#4 @ 11.5	#4 @ 12.5
4'-9"	8	#4 @ 8	#4 @ 8.5	#4 @ 9	#4 @ 9.5	#4 @ 11	#4 @ 12.5
5'-0"	8	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 8.5	#4 @ 10.5	#4 @ 12.5
5'-3"	8	#4 @ 6.5	#4 @ 7	#4 @ 7.5	#4 @ 8	#4 @ 10	#4 @ 12
5'-6"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 7.5	#4 @ 9.5	#4 @ 12
5'-9"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 9	#4 @ 11.5
6'-0"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5	#4 @ 11
6'-3"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5	#4 @ 11
6'-6"	8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8	#4 @ 10.5
6'-9"	8	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5	#4 @ 10.5
7'-0"	8	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7	#4 @ 10
7'-3"	8	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 10
7'-6"	8	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 6.5	#4 @ 9.5
7'-9"	8	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8.5

8'-0"	8	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#4 @ 6	#4 @ 6.5	#4 @ 8
8'-3"	8.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8
8'-6"	8.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 8
8'-9"	8.5	#5 @ 7.5	#5 @ 7.5	#4 @ 6.5	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5
9'-0"	8.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7.5
9'-3"	8.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#4 @ 6.5	#4 @ 6.5	#4 @ 7
9'-6"	9	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 8.5	#5 @ 9.5	#4 @ 7
9'-9"	9	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 8	#5 @ 9.5	#4 @ 7
10'-0"	9	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 9	#4 @ 6.5
10'-3"	9	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9.5
10'-6"	9	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5	#5 @ 9
10'-9"	9.5	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9
11'-0"	9.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8.5	#5 @ 9
11'-3"	9.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 9
11'-6"	9.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 9
11'-9"	10	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 7.5	#5 @ 8	#5 @ 9
12'-0"	10	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 9
12'-3"	10	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8.5
12'-6"	10	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
12'-9"	10.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8	#5 @ 8.5
13'-0"	10.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8.5
13'-3"	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
13'-6"	10.5	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8.5
13'-9"	11	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7	#5 @ 8	#5 @ 8
14'-0"	11	#5 @ 6.5	#5 @ 6.5	#5 @ 7	#5 @ 7	#5 @ 7.5	#5 @ 8

Table 17.10-2

Transverse Reinforcing Steel for Deck Slabs on Precast Concrete Deck Panels

Notes:

- Designed per AASHTO LRFD with HL-93 Loading.
- f’c deck = 4.0 ksi
- fy = 60 ksi
- Steel is 2 ½” clear from top of slab. Designed for 20 psf future wearing surface. “Total Slab Thickness” includes thickness of deck panel and poured in place concrete.

Table of Contents

18.1 Introduction

18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab
- Haunched Slab

A longitudinal slab is one of the least complex types of bridge superstructures. It is composed of a single element superstructure in comparison to the two elements of the transverse slab on girders or the three elements of a longitudinal slab on floor beams supported by girders. Due to simplicity of design and construction, the concrete slab structure is relatively economical. Its limitation lies in the practical range of span lengths and maximum skews for its application. For longer span applications, the dead load becomes too high for continued economy. Application of the haunched slab has increased the practical range of span lengths for concrete slab structures.

18.1.2 Limitations

Concrete slab structure types are not recommended over streams where the normal water freeboard is less than 4 feet; formwork removal requires this clearance. When spans exceed 35 feet, freeboard shall be increased to 5 feet above normal water.

All concrete slab structures are limited to a maximum skew of 30 degrees. Slab structures with skews in excess of 30 degrees, require analysis of complex boundary conditions that exceed the capabilities of the present design approach used in the Bureau of Structures.

Continuous span slabs are to be designed using the following pier types:

- Piers with pier caps (on columns or shafts)
- Wall type piers

These types will allow for ease of future superstructure replacement. Piers that have columns without pier caps, have had the columns damaged during superstructure removal. This type of pier will not be allowed without the approval of the Structures Design Section.

WisDOT policy item:

Slab bridges, due to camber required to address future creep deflection, do not ride ideally for the first few years of their service life and present potential issues due to ponding. As such, if practical (e.g. not excessive financial implications), consideration of other structure types should be given for higher volume/higher speed facilities, such as the Interstate. Understanding these issues, the Regions have the responsibility to make the final decision on structure type with respect to overall project cost, with BOS available for consultation.

18.2 Specifications, Material Properties and Structure Type

18.2.1 Specifications

Reference may be made to the design and construction related material as presented in the following specifications:

- *State of Wisconsin, Department of Transportation Standard Specifications for Highway and Structure Construction*

  Section 502 - Concrete Bridges

  Section 505 - Steel Reinforcement

- Other Specifications as referenced in Chapter 3

18.2.2 Material Properties

The properties of materials used for concrete slab structures are as follows:

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

4 ksi, for concrete slab superstructure

3.5 ksi, for concrete substructure units

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in slab **LRFD [C5.4.2.4]**

= $33{,}000\ K_1\ w_c^{1.5}\ (f'_c)^{1/2} = 3800$ ksi

Where:

K_1 = 1.0

w_c = 0.150 kcf, unit weight of concrete

n = $E_s / E_c = 8$ **LRFD [5.7.1]** (modular ratio)

18.2.3 Structure Type and Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, approximate slab depth, skew, roadway width, etc.. The selection of the type of concrete slab structure (haunched / flat) is a function of the span lengths selected. Recommended span length ranges and corresponding structure type are shown for single-span and multiple-span slabs in Figure 18.2-1. Estimated slab depths are shown in Table 18.2-1.

Currently, voided slab structures are not allowed. Some of the existing voided slabs have displayed excessive longitudinal cracking over the voids in the negative zone. This may have been caused by the voids deforming or floating-up due to lateral pressure during the concrete pour. Recent research indicates slabs with steel void-formers have large crack widths above the voids due to higher stress concentrations.

If optimum span ratios are selected such that the positive moments in each span are equal, the interior and end span slab depths will be equal, provided Strength Limit State controls. Optimum span ratios are independent of applied live loading.

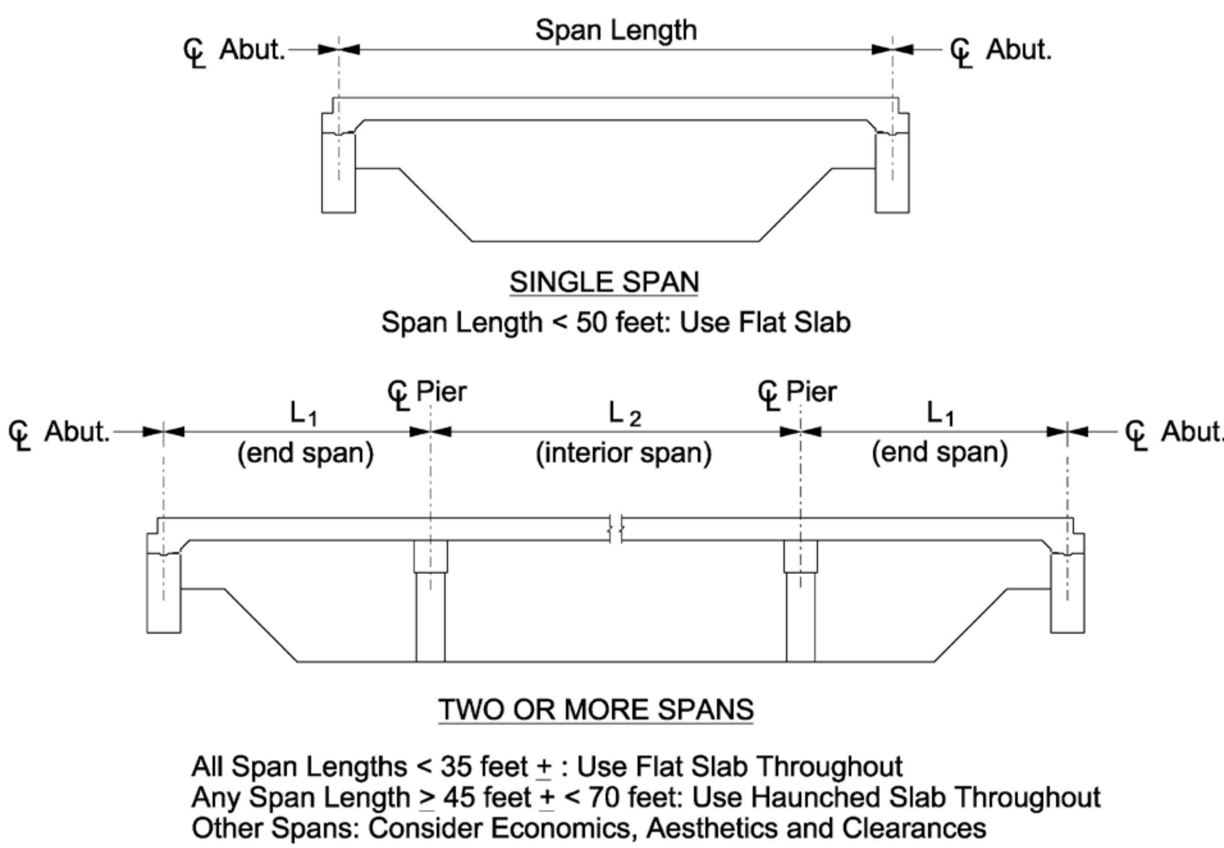


Figure 18.2-1
Span Length vs. Slab Type

For the following optimum span ratio equations based on Strength Limit State controlling, L_1 equals the end span lengths and L_2 equals the interior span length or lengths, for structures with three or more spans.

For flat slabs the optimum span ratio is obtained when $L_2 = 1.25L_1$. The optimum span ratio for a three-span haunched slab results when $L_2 = L_1(1.43 - 0.002L_1)$ and for a four-span haunched slab when $L_2 = 1.39L_1$.

Approximate slab depths for multiple-span flat and haunched slabs can be obtained from Table 18.2-1. These values are to be used for dead load computations and preliminary computations only and the final slab depth is to be determined by the designer.

(s) Span Length (feet)	Slab Depth (inches)	
	Haunched [1]	Flat [4]
20	---	12
25	---	14
30	---	16
35	---	18
40	---	20
45	16 [2]	22
50	17.5 [2]	24
55	19 [2]	26
60	20 [2]	---
65	22 [3]	---
70	25 [3]	---

Table 18.2-1
Span Length vs. Slab Depth

[1] These estimated slab depths at mid-span, apply to interior spans of three or more span structures, with an end span length of approximately 0.7 times the interior span. Depths are based on dead load deflection (camber) and live load deflection limits. Haunch length (L_{haunch}) = 0.167 (L_2), and d_{slab} / D_{haunch} = 0.6 were used. L_2 = interior span length, (d_{slab}) = slab depth in span and (D_{haunch}) = slab depth at haunch. Values in table include ½ inch wearing surface.

[2] Depths controlled by live load deflection criteria

[3] Depths controlled by dead load deflection (camber) criteria

[4] These values represent **LRFD [2.5.2.6.3]** recommended minimum depths for continuous-spans using (s+10)/30. The slab span length (s) in the equation and resulting minimum depths are in feet and are presented in inches in Table 18.2-1. For simple-spans, the Bureau of Structures adds 10% greater depth and checks the criteria in 18.4.4. Values in table include ½ inch wearing surface.

The minimum slab depth is 12 inches. Use increments of ½ inch to select depths > 12 inches.

18.3 Limit States Design Method

18.3.1 Design and Rating Requirements

All new concrete slab structures are to meet design requirements as stated in 17.1.1 and rating requirements as stated in 17.1.2.

18.3.2 LRFD Requirements

18.3.2.1 General

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: **LRFD [1.3.2.1, 5.5.1]**

$$Q = \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$ (Limit States Equation) **LRFD [1.3.2.1, 3.4.1]**

Where:

η_i = load modifier (a function of η_D , η_R and η_I) **LRFD [1.3.2.1, 1.3.3, 1.3.4, 1.3.5]**

γ_i = load factor

Q_i = force effect; moment, shear, stress range or deformation caused by applied loads

Q = total factored force effect

ϕ = resistance factor

R_n = nominal resistance; resistance of a component to force effects

R_r = factored resistance = ϕR_n

The Limit States used for concrete slab design are:

- Strength I Limit State
- Service I Limit State
- Fatigue I Limit State

18.3.2.2 Statewide Policy

Current Bureau of Structures policy is :

- Set value of load modifier, η_i , and its factors (η_D , η_R , η_I) all equal to 1.00 for concrete slab design.

- Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects, Q_i , for slab bridges.

- Ignore reduction factor, r, for skewed slab bridges in **LRFD [4.6.2.3]** that would reduce longitudinal force effects, Q_i .

18.3.3 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a bridge is expected to experience during its design life **LRFD [1.3.2.4]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Strength I Limit State **LRFD [3.4.1]** will be used for:

- Designing longitudinal slab reinforcement for flexure

- Designing transverse slab reinforcement over the piers for flexure

- Checking shear (two-way) in slab at the piers

- Checking uplift at the abutments

- Checking longitudinal slab reinforcement for tension from shear

18.3.3.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Strength I Limit State will be used to design the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, defined in 18.4.3.2.

The load factor, γ_i , is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

For Strength I Limit State, the values of γ_i for each applied load, are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]** and their values are: γ_{DC} = 1.25/0.90, γ_{DW} = 1.50/0.65, $\gamma_{LL+IM} = \gamma_{PL}$ = 1.75. The values for γ_{DC} and γ_{DW} have a maximum and minimum value.

Therefore, for Strength I Limit State:

$$Q = 1.0\,[\,1.25(DC) + 1.50(DW) + 1.75((LL + IM) + PL)\,]$$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads. The load factors shown for DC and DW are maximum values. Use maximum or minimum values as shown in **LRFD [Table 3.4.1-2]** to calculate the critical force effect.

18.3.3.2 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for Strength Limit State **LRFD [5.5.4.2]** are:

- ϕ = 0.90 for flexure & tension (for tension-controlled reinforced concrete sections as defined in **LRFD [5.7.2.1]**)

- ϕ = 0.90 for shear and torsion

The factored resistance, R_r (M_r, V_r, T_{cap}), associated with the list of items to be designed/checked using Strength I Limit State in 18.3.3, are described in the following sections.

18.3.3.2.1 Moment Capacity

Stress is assumed proportional to strain below the proportional limit on the stress-strain diagram. Tests have shown that at high levels of stress in concrete, stress is not proportional to strain. Recognizing this fact, strength analysis takes into account the nonlinearity of the stress-strain diagram. This is accomplished by using a rectangular stress block to relate the concrete compressive stress distribution to the concrete strain. The compressive stress block has a uniform value of $\alpha_1 \cdot f'_C$ over a zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance a = $\beta_1 \cdot (c)$ from the extreme compression fiber. The distance (c) is measured perpendicular to the neutral axis. The factor α_1 shall be taken as 0.85 for concrete strengths not exceeding 10.0 ksi and the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi **LRFD [5.7.2.2]**. Strength predictions using this method are in agreement with strength test results. The representation of these assumptions is shown in Figure 18.3-1.

The moment capacity (factored resistance) of concrete components shall be based on the conditions of equilibrium and strain compatibility, resistance factors as specified in **LRFD [5.5.4.2]** and the assumptions outlined in **LRFD [5.7.2]**.

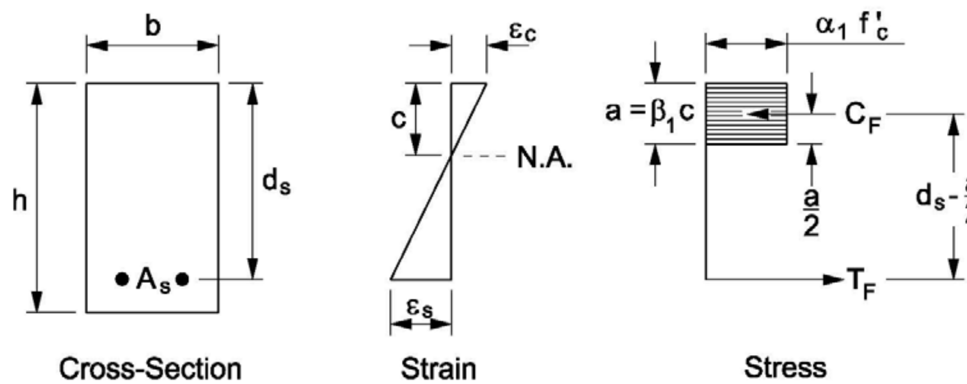


Figure 18.3-1
Stress / Strain on Cross - Section

Referring to Figure 18.3-1, the internal force equations are:

$C_F = \alpha_1 \cdot (f'_c)\ (b)\ (a) = 0.85\ (f'_c)\ (b)\ (a)$

$T_F = (A_s)\ (f_s)$

By equating C_F to T_F, and solving for the compressive stress block depth, (a), gives:

$a = A_s\ f_s\ /\ 0.85\ (f'_c)\ (b)$

Use ($f_s = f_y$) when the steel yields prior to crushing of the concrete. To check for yielding, assume ($f_s = f_y$) and calculate the value for (a). Then calculate the value for $c = a / \beta_1$ and d_s as shown in Figure 18.3-1. If c / d_s does not exceed the value calculated below, then the reinforcement has yielded and the assumption is correct, as stated in **LRFD [5.7.2.1]**.

$c / d_s \leq 0.003 / (0.003 + \varepsilon_{cl})$

ε_{cl} = compression controlled strain limit

for f_y = 60 ksi, ε_{cl} is 0.0020 per **LRFD [Table C5.7.2.1-1]**

if $c / d_s \leq 0.6$, then the reinforcement (f_y = 60 ksi) will yield and ($f_s = f_y$)

For rectangular sections, the nominal moment resistance, M_n , (tension reinforcement only) equals: **LRFD [5.7.3.2.3]**

$M_n = A_s\ f_s\ (d_s – a/2)$

The factored resistance, M_r , or moment capacity, shall be taken as: **LRFD [5.7.3.2.1]**

$M_r = \phi\ M_n = \phi\ A_s\ f_s\ (d_s – a/2)$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$$M_r = (0.9)\, A_s\, f_s\, (d_s – a/2)$$

18.3.3.2.2 Shear Capacity

The nominal shear resistance, V_n , for two-way action, shall be determined as: **LRFD [5.8.1.4, 5.13.3.6.3]**

$$V_n = (0.063 + 0.126 / \beta_c)\, \lambda\, (f'_c)^{1/2}\, b_o\, d_v \leq 0.126\, \lambda\, (f'_c)^{1/2}\, b_o\, d_v \quad \text{(kips)}$$

Where:

f'_c	=	4.0 ksi (for concrete slab bridges)
β_c	=	ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted
d_v	=	effective shear depth as determined in **LRFD [5.8.2.9]** (in)
b_o	=	perimeter of the critical section (in)
λ	=	conc. density modification factor ; for normal weight conc. = 1.0 , **LRFD [5.4.2.8]**

The factored resistance, V_r , or shear capacity, shall be taken as: **LRFD [5.8.2.1]**

$$V_r = \phi\, V_n$$

The resistance factor, ϕ, is 0.90, therefore:

$$V_r = (0.9)\, V_n$$

18.3.3.2.3 Uplift Check

The check of uplift at abutments does not use a factored resistance, but compares factored dead load and live load reactions.

18.3.3.2.4 Tensile Capacity – Longitudinal Reinforcement

The nominal tensile resistance, T_{nom} , for an area, A_s , of developed reinforcement, equals:

$$T_{nom} = A_s\, f_y$$

The factored resistance, T_{cap} , or tensile capacity, shall be taken as:

$$T_{cap} = \phi\, T_{nom} = \phi\, A_s\, f_y$$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$T_{cap} = (0.9)\ A_s\ f_y$

18.3.4 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Service I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for crack control criteria
- Checking transverse slab reinforcement over the piers for crack control criteria
- Checking live load deflection criteria
- Checking dead load deflection (camber) criteria

18.3.4.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Service I Limit State will be used to analyze the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and/or appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, where applicable, defined in 18.4.3.2.

For Service I Limit State, the values of γ_i for each applied load, are found in **LRFD [Table 3.4.1-1]** and their values are: $\gamma_{DC} = \gamma_{DW} = \gamma_{LL+IM} = \gamma_{PL} = 1.0$

Therefore, for Service I Limit State:

$$Q = 1.0\ [\ 1.0(DC) + 1.0(DW) + 1.0((LL + IM) + PL)\ ]$$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads.

18.3.4.2 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

The factored resistance, R_r , associated with the list of items to be checked using Service I Limit State in 18.3.4, are described in the following sections.

18.3.4.2.1 Crack Control Criteria

All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

18.3.4.2.2 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits. The Bureau of Structures limits live load deflections for concrete slab structures to L/1200. The deflections are based on entire slab width acting as a unit and gross moment of inertia, I_g .

The nominal resistance, R_n , or deflection limit, is:

$R_n = L/1200$

Where:

L = span length

The factored resistance, R_r , is:

$R_r = \phi\ R_n = \phi\ (L/1200)$

The resistance factor, ϕ, is 1.00, therefore:

$R_r = (1.0)\ R_n = (L/1200)$

18.3.4.2.3 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g . Bureau of Structures calculates full camber based on multiplying the dead load deflection values by a factor of three. A maximum allowable camber has been set for simple-span slabs and continuous-span slabs as shown in 18.4.4.2.

The nominal resistance, R_n , or deflection limit, is:

R_n = (maximum allowable camber) / 3

The factored resistance, R_r , is:

$R_r = \phi\ R_n = \phi$ (maximum allowable camber) / 3

The resistance factor, ϕ, is 1.00, therefore:

$R_r = (1.0)\ R_n$ = (maximum allowable camber) / 3

18.3.5 Fatigue Limit State

Fatigue I Limit State shall be applied as a restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles **LRFD [1.3.2.3]**. The Fatigue I Limit State is intended to limit crack growth under repetitive loads to prevent fracture of the reinforcement during the design life of the bridge. The factored force effect (stress range), Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

For fatigue considerations, concrete members shall satisfy: **LRFD [5.5.3.1]**

$$\eta_i\ \gamma_i\ (\Delta f) \leq (\Delta F)_{TH}$$

Where:

γ_i	=	Load factor for Fatigue I Limit State
Δf	=	Force effect, live load stress range due to the passage of the fatigue truck (ksi)
$(\Delta F)_{TH}$	=	Constant-amplitude fatigue threshold (ksi)

Fatigue I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for fatigue stress range criteria

18.3.5.1 Factored Loads (Stress Range)

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Fatigue I Limit State will be used to analyze the structure for force effects, $Q_i = (\Delta f)$, due to applied (Fatigue Truck) live load, LL and IM, defined in 18.4.3.1.

For Fatigue I Limit State, the value of γ_i for the applied live load, is found in **LRFD [Table 3.4.1-1]** and its value is $\gamma_{LL+IM} = 1.5$.

Therefore, for Fatigue I Limit State:

$$Q = 1.0\ [\ 1.5(LL + IM)\ ]$$

Where LL and IM represent force effects, Δf, due to these applied loads.

18.3.5.2 Factored Resistance

The resistance factor, ϕ, for Fatigue Limit State, is found in **LRFD [C1.3.2.1]** and its value is 1.00 .

18.3.5.2.1 Fatigue Stress Range

The nominal resistance, $R_n = (\Delta F)_{TH}$, for fatigue stress range (for straight reinforcement), is: **LRFD [5.5.3.2]**

$$R_n = (\Delta F)_{TH} = 24 - 20\, f_{min} / f_y \quad \text{(ksi)}$$

Where:

f_{min}	=	the minimum stress resulting from the factored Fatigue Truck live load, combined with the stress from the dead loads on the structure; positive if tension, negative if compression (ksi)
f_y	=	minimum yield strength (ksi), not to be taken less than 60 ksi nor greater than 100 ksi

The factored resistance, R_r (for f_y = 60 ksi), is:

$$R_r = \phi\, R_n = \phi\, (24 - 0.33\, f_{min})$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0)\, R_n = 24 - 0.33\, f_{min} \quad \text{(ksi)}$$

18.4 Concrete Slab Design Procedure

18.4.1 Trial Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, skew, roadway width, etc.. The selection of the type of concrete slab structure (haunched / flat) is a function of the span lengths selected. Recommended span length ranges and corresponding structure type are shown for single-span and multiple-span slabs in Figure 18.2-1. Optimum span ratios for multiple-span slabs are suggested in 18.2.3. Knowing the span lengths and the structure type, a trial slab depth can be obtained from Table 18.2-1.

For haunched slabs, the haunch depth, D_{haunch} , is proportional to the slab depth, d_{slab} , outside the haunch. A trial haunch depth can be selected as:

$$D_{haunch} = d_{slab} / 0.6$$

An economical haunch length, L_{haunch} , measured from C/L of pier to end of haunch, can be approximated between ($0.15\ L_2$ to $0.18\ L_2$), where L_2 is the length of an interior span.

NOTE: With preliminary structure sizing complete, check to see if structure exceeds limitations in 18.1.2.

18.4.2 Dead Loads (DC, DW)

Dead loads (permanent loads) are defined in **LRFD [3.3.2]**. Concrete dead load is computed by using a unit weight of 150 pcf, with no adjustment in weight for the bar steel reinforcement.

DC	=	dead load of structural components and any nonstructural attachments
DW	=	dead load of future wearing surface (F.W.S.) and utilities

The slab dead load, DC_{slab} , and the section properties of the slab, do not include the ½ inch wearing surface. A post dead load, DW_{FWS} , of 20 psf, for possible future wearing surface (F.W.S.), is required in the design by the Bureau of Structures. The ½ inch wearing surface load, $DC_{1/2"\ WS}$, of 6 psf must also be included in the design of the slab.

Dead loads, DC, from parapets, medians and sidewalks are uniformly distributed across the full width of the slab when designing an interior strip. For the design of exterior strips (edge beams), any of these dead loads, DC, that are located directly over the exterior strip width and on the cantilevered portion of sidewalks, shall be applied to the exterior strip. For both interior and exterior strips, the future wearing surface, DW, located directly over the strip width shall be applied to it. See 17.2.7 for the distribution of dead loads.

18.4.3 Live Loads

18.4.3.1 Vehicular Live Load (LL) and Dynamic Load Allowance (IM)

The *AASHTO LRFD* Specifications contain several live load components (see 17.2.4.2) that are combined and scaled to create live load combinations that apply to different Limit States **LRFD [3.6.1]**.

The live load combinations used for design are:

LL#1:	Design Tandem (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#2:	Design Truck (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#3:	90% [Double Design Trucks (+ IM) + Design Lane Load]	**LRFD [3.6.1.3.1]**
LL#4:	Fatigue Truck (+ IM)	**LRFD [3.6.1.4.1]**
LL#5:	Design Truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6:	25% [Design Truck (+ IM)] + Design Lane Load	**LRFD [3.6.1.3.2]**

Table 18.4-1
Live Load Combinations

The dynamic load allowance, IM, **LRFD [3.6.2]** for the live load combinations above, is shown in Table 18.4-2.

Where (IM) is required, multiply the loads by (1 + IM/100) to include the dynamic effects of the load. (IM) is not applied to the Design Lane Load.

The live load combinations are applied to the Limit States as shown in Table 18.4-2.

The live load force effect, Q_i , shall be taken as the largest from the live loads shown in Table 18.4-2 for that Limit State.

Strength I Limit State: [1]	LL#1 , LL#2 , LL#3 [2]	IM = 33%
Service I Limit State: [1] (for crack control criteria)	LL#1 , LL#2 , LL#3 [2]	IM = 33%
Service I Limit State: (for LL deflection criteria)	LL#5 , LL#6	IM = 33%
Fatigue I Limit State: [3]	LL#4 (single Fatigue Truck)	IM = 15%

Table 18.4-2
Live Loads for Limit States

[1] Load combinations shown are used for design of interior strips and exterior strips without raised sidewalks, as shown in Figures 17.2-6 to 10. For an exterior strip with a raised sidewalk,

use Design Lane Load portion of LL#2 for Live Load Case 1 and use Design Truck (+IM) portion of LL#2 for Live Load Case 2, as shown in Figure 17.2-11.

[2] (LL#3) is used to calculate negative live load moments between points of contraflexure and also reactions at interior supports. The points of contraflexure are located by placing a uniform load across the entire structure. For these moments and reactions, the results calculated from (LL#3) are compared with (LL#1) and (LL#2) results, and the critical value is selected.

[3] Used for design of interior strip only.

18.4.3.2 Pedestrian Live Load (PL)

For bridges designed for both vehicular and pedestrian live load, a pedestrian live load, PL, of 75 psf is used. However, for bridges designed exclusively for pedestrian and/or bicycle traffic, see *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* for live load. The dynamic load allowance, IM, is not applied to pedestrian live loads **LRFD [3.6.2]**.

Pedestrian loads are not applied to an interior strip for its design. For the design of exterior strips (edge beams), any pedestrian loads that are located directly over the exterior strip width and on the cantilevered portion of the sidewalk, shall be applied to the exterior strip. See 17.2.7 for the distribution of pedestrian live loads.

18.4.4 Minimum Slab Thickness Criteria

Check adequacy of chosen slab thickness by looking at live load deflection and dead load deflection (camber) criteria, using Service I Limit State.

18.4.4.1 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2]**. Live load deflections for concrete slab structures are limited to L/1200, by the Bureau of Structures. The live load deflection, Δ_{LL+IM} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.3.1 for Service I Limit State.

Place live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**. Use gross moment of inertia, I_g , based on entire slab width acting as a unit. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2. The factored resistance, R_r , is described in 18.3.4.2.2.

Then check that, $\Delta_{LL+IM} \leq R_r$ is satisfied.

18.4.4.2 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits **LRFD [5.7.3.6.2]**. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g . All dead loads are to be uniformly distributed across the width of the slab. These deflections are increased to provide for the time-dependent deformations of creep and shrinkage. Bureau of Structures currently calculates full camber as three times the dead load deflection. Most of the excess camber is dissipated during the first year of service, which is the time period that the majority of creep and shrinkage deflection occurs. Noticeable excess deflection or structure sag can normally be attributed to falsework settlement. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2 . The dead load deflection, Δ_{DL} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.2. The factored resistance, R_r , is described in 18.3.4.2.3.

WisDOT exception to AASHTO:

Calculating full camber as three times the dead load deflection, as stated in paragraph above, is an exception to **LRFD [5.7.3.6.2]**. This exception, used by the Bureau of Structures, is based on field observations using this method.

Then check that, $\Delta_{DL} \leq R_r$ is satisfied.

A "Camber Diagram" is shown in the plans on the "Superstructure" sheet. Provide camber values, as well as centerline and edge of slab elevations, at 0.1 points of all spans.

Simple-Span Concrete Slabs:

Maximum allowable camber for simple-span slabs is limited to 2 ½ inches. For simple-span slabs, Bureau of Structures practice indicates that using a minimum slab depth (ft) from the equation 1.1(S + 10) / 30, (where S is span length in feet), and meeting the live load deflection and dead load deflection (camber) limits stated in this section, provides an adequate slab section for most cases.

WisDOT exception to AASHTO:

The equation for calculating minimum slab depth for simple-spans, as stated in paragraph above, is an exception to **LRFD [Table 2.5.2.6.3-1]**. This exception, used by the Bureau of Structures, is based on past performance using this equation.

Continuous-Span Concrete Slabs:

Maximum allowable camber for continuous-span slabs is 1 ¾ inches.

18.4.5 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below. The equivalent distribution width applies for both live load moment and shear.

18.4.5.1 Interior Strip

Equivalent interior strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3]**.

The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load.

Single-Lane Loading: $E = 10.0 + 5.0\,(L_1 W_1)^{1/2}$

Multi-Lane Loading: $E = 84.0 + 1.44(L_1 W_1)^{1/2} \leq 12.0(W)/N_L$

Where:

E = equivalent distribution width (in)

L_1 = modified span length taken equal to the lesser of the actual span or 60.0 ft (ft)

W_1 = modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60.0 ft for multi-lane loading, or 30.0 ft for single-lane loading (ft)

W = physical edge to edge width of bridge (ft)

N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

18.4.5.1.1 Strength and Service Limit State

Use the smaller equivalent width (single-lane or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E}$$

Where:

E = equivalent distribution width (ft)

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore aren’t used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.5.1.2 Fatigue Limit State

Use equivalent widths from single-lane loading to check fatigue stress range criteria. For the Fatigue Limit State only one design truck (Fatigue Truck) is present **LRFD [3.6.1.4]**. Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor, m, which are present in the equation for equivalent width, E, **LRFD [3.6.1.1.2]**.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E(1.20)}$$

Where:

E = equivalent distribution width (ft)

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.5.2 Exterior Strip

Equivalent exterior strip widths for slab bridges are covered in **LRFD [4.6.2.1.4]**.

For Exterior Strips without Raised Sidewalks:

The exterior strip width, E, is assumed to carry one wheel line and a tributary portion of design lane load (located directly over the strip width) as shown in Figures 17.2-7 and 17.2-9.

E equals the distance between the edge of the slab and the inside face of the barrier, plus 12 inches, plus ¼ of the full strip width specified in **LRFD [4.6.2.3]**.

The exterior strip width, E, shall not exceed either ½ the full strip width or 72 inches.

Use the smaller equivalent width (single-lane or multi-lane), for full strip width, when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The multiple presence factor, m, has been included in the equations for full strip width and therefore aren't used to adjust the distribution factor **LRFD [3.6.1.1.2]**.

For Exterior Strips with Raised Sidewalks:

The exterior strip width, E, is to carry a tributary portion of design lane load (when its located directly over the strip width) as in Live Load Case 1 or one wheel line as in Live Load Case 2, as shown in Figure 17.2-11.

The exterior strip width, E, shall be 72 inches.

18.4.5.2.1 Strength and Service Limit State

The distribution factor, DF, is computed for a design slab width equal to one foot.

Compute the distribution factor associated with one truck wheel line, to be applied to axle loads:

$$DF = \frac{(1 \text{ wheel line})}{(2 \text{ wheel lines/lane})(E)}$$

Where:

E = equivalent distribution width (ft)

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to full lane load: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\dfrac{(SWL)}{(10\text{ ft lane load width})}\right]}{(E)}$$

Where:

E = equivalent distribution width (ft)

SWL = Slab Width Loaded (with lane load) (ft) $\geq$ 0.

E – (distance from edge of slab to inside face of barrier) or

E – (distance from edge of slab to inside face of raised sidewalk)

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.6 Longitudinal Slab Reinforcement

The concrete cover on the top bars is 2 ½ inches, which includes a ½ inch wearing surface. The bottom bar cover is 1 ½ inches. Minimum clear spacing between adjacent longitudinal bars is 3 ½ inches. The maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the slab or 18.0 inches **LRFD [5.10.3.2]**. When bundled bars are used, see **LRFD [5.10.3.1.5, 5.11.2.3, 5.11.5.2.1]**.

18.4.6.1 Design for Strength

Strength Limit State considerations and assumptions are detailed in **LRFD [5.5.4, 5.7.2]**.

The area of longitudinal slab reinforcement, A_s , should be designed for strength at maximum moment locations along the structure, and for haunched slab structures, checked for strength at the haunch/slab intercepts. The area should also be checked for strength at bar reinforcement cutoff locations. This reinforcement should be designed for interior and exterior strips (edge beams) in both positive and negative moment regions. The reinforcement in the exterior strip is always equal to or greater than that required for the slab in an interior strip. Compare the reinforcement to be used for each exterior strip and select the strip with the largest amount of reinforcement (in^2/ft). Use this reinforcement pattern for both exterior strips to keep the bar layout symmetrical. Concrete parapets, curbs, sidewalks and other appurtenances are not to be considered to provide strength to the edge beam **LRFD [9.5.1]**. The total factored moment, M_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Then calculate the coefficient of resistance, R_u :

$$R_u = M_u / \phi \, b \, d_s^2$$

Where:

ϕ	=	0.90 (see 18.3.3.2)
b	=	12 in (for a 1 foot design slab width)
d_s	=	slab depth (excl. ½ inch wearing surface) – bar clearance – ½ bar diameter (in)

Calculate the reinforcement ratio, ρ, using (R_u vs. ρ) Table 18.4-3 .

Then calculate required area,

$$A_s = \rho \, (b) \, (d_s)$$

Area of bar reinforcement per foot of slab width can be found in Table 18.4-4 .

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_u \leq M_r$ is satisfied.

The area of longitudinal reinforcement, A_s , should also be checked for moment capacity (factored resistance) along the structure, to make sure it can handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1. See Chapter 45 for details on checking the capacity of the structure for this Permit Vehicle.

18.4.6.2 Check for Fatigue

Fatigue Limit State considerations and assumptions are detailed in **LRFD [5.5.3, 5.7.1, 9.5.3]**

The area of longitudinal slab reinforcement, A_s , should be checked for fatigue stress range at locations where maximum stress range occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for fatigue stress range at bar reinforcement cutoff locations using Fatigue I Limit State. Check the reinforcement in an interior strip, where the largest number of fatigue cycles will occur.

Fatigue life of reinforcement is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

In regions where stress reversal takes place, continuous concrete slabs will be doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered.

In regions of compressive stress due to permanent loads, fatigue shall be considered only if this compressive stress is less than 1.5 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.5 times the fatigue load is tensile and exceeds $0.095\ (f'_c)^{1/2}$.

The factored stress range, Q, shall be calculated using factored loads described in 18.3.5.1. The factored resistance, R_r , shall be calculated as in 18.3.5.2.1.

Then check that, Q (factored stress range) $\leq R_r$ is satisfied.

Reference is made to the design example in 18.5 of this chapter for computations relating to reinforcement remaining in tension throughout the fatigue cycle, or going through tensile and compressive stresses during the fatigue cycle.

18.4.6.3 Check for Crack Control

Service Limit State considerations and assumptions are detailed in **LRFD [5.5.2, 5.7.1, 5.7.3.4]**.

The area of longitudinal slab reinforcement, A_s , should be checked for crack control at locations where maximum tensile stress occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for crack control at bar reinforcement cutoff locations using Service I Limit State. Check the reinforcement in an interior and exterior strip (edge beam).

The use of high-strength steels and the acceptance of design methods where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over the bars, and spacing of reinforcement. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r , specified in **LRFD [5.4.2.6]**, for Service I Limit State. The spacing of reinforcement, s, in the layer closest to the tension face shall satisfy:

$s \leq (700\ \gamma_e / \beta_s f_{ss}) - 2\ (d_c)$ (in)

Bar spacing, s, need not be less than 5 in. for control of flexural cracking **LRFD [5.7.3.4]**

in which:

$\beta_s = 1 + (d_c) / 0.7\ (h - d_c)$

Where:

γ_e = 1.00 for Class 1 exposure condition (bottom reinforcement)

γ_e = 0.75 for Class 2 exposure condition (top reinforcement)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto, (in). For calculation purposes, d_c need not be taken greater than 2 in. plus the bar radius

f_{ss} = tensile stress in steel reinforcement (ksi) $\leq 0.6f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate (f_{ss})

h = overall depth of the section (in)

18.4.6.4 Minimum Reinforcement Check

The area of longitudinal slab reinforcement, A_s , should be checked for minimum reinforcement requirement at locations along the structure **LRFD [5.7.3.3.2]**.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , or moment capacity, at least equal to the lesser of:

M_{cr} (or) 1.33 M_u

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

f_r = $0.24 \lambda (f'c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]**

γ_1 = 1.6 flexural cracking variability factor

γ_3 = 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement

I_g = gross moment of Inertia (in^4)

c = effective slab thickness/2 (in)

M_u = total factored moment, calculated using factored loads described in 18.3.3.1 for Strength I Limit State

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Select lowest value of [M_{cr} (or) 1.33 M_u] = M_L

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_L \leq M_r$ is satisfied.

18.4.6.5 Bar Cutoffs

One-half of the bar steel reinforcement required for maximum moment can be cut off at a point, where the remaining one-half has the moment capacity, or factored resistance, M_r , equal to the total factored moment, M_u , at that point. This is called the theoretical cutoff point.

Select tentative cutoff point at theoretical cutoff point or at a distance equal to the development length from the point of maximum moment, whichever is greater. The reinforcement is extended beyond this tentative point for a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. This cutoff point is acceptable, if it satisfies fatigue and crack control criteria. The continuing bars must be fully developed at this point **LRFD [5.11.1.2.1]**.

18.4.6.5.1 Positive Moment Reinforcement

At least one-third of the maximum positive moment reinforcement in simple-spans and one-fourth of the maximum positive moment reinforcement in continuous-spans is extended along the same face of the slab beyond the centerline of the support **LRFD [5.11.1.2.2]**.

18.4.6.5.2 Negative Moment Reinforcement

For negative moment reinforcement, the second tentative cutoff point is at the point of inflection. At least one-third of the maximum negative moment reinforcement must extend beyond this point for a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater **LRFD [5.11.1.2.3]**.

18.4.7 Transverse Slab Reinforcement

18.4.7.1 Distribution Reinforcement

Distribution reinforcement is placed transversely in the bottom of the slab, to provide for lateral distribution of concentrated loads **LRFD [5.14.4.1]**. The criteria for main reinforcement parallel to traffic is applied. The amount of distribution reinforcement is to be determined as a percentage of the main reinforcing steel required for positive moment as given by the following formula:

$$\text{Percentage} = \frac{100\%}{\sqrt{L}} \leq 50\% \text{ maximum}$$

Where:

L = span length (ft)

The above formula is conservative when applied to slab structures. This specification was primarily drafted for the relatively thin slabs on stringers.

18.4.7.2 Reinforcement in Slab over Piers

If the concrete superstructure rests on a pier cap (with columns) or directly on columns, design of transverse slab reinforcement over the pier is required. A portion of the slab over the pier is designed as a continuous transverse slab member (beam) along the centerline of the substructure. The depth of the assumed section is equal to the depth of the slab or haunch when the superstructure rests directly on columns. When the superstructure rests on a pier cap and the transverse slab member and pier cap act as a unit, the section depth will include the slab or haunch depth plus the cap depth. For a concrete slab, the width of the transverse slab member is equal to one-half the center to center spacing between columns (or 8 foot maximum) for the positive moment zone. The width equals the diameter of the column plus 6 inches for negative moment zone when no pier cap is present. The width equals the cap width for negative moment zone when a pier cap is present. Reference is made to the design example in 18.5 of this chapter for computations relating to transverse reinforcement in slab over the piers.

18.4.8 Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete.

The area, A_s , of reinforcement per foot for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8]**

$A_s \geq 1.30\ (b)\ (h)\ /\ 2\ (b+h)\ (f_y)$ and $0.11 \leq A_s \leq 0.60$

Where:

A_s	=	area of reinforcement in each direction and on each face (in^2/ft)
b	=	least width of component section (in)
h	=	least thickness of component section (in)
f_y	=	specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches. For components greater than 36 inches thick, the spacing shall not exceed 12 inches.

All longitudinal reinforcement and transverse reinforcement in the slab must exceed required A_s (on each face and in each direction), and not exceed maximum spacing.

18.4.9 Shear Check of Slab

Slab bridges designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.14.4.1]**.

18.4.10 Longitudinal Reinforcement Tension Check

The tensile capacity check of longitudinal reinforcement on the flexural tension side of a member is detailed in **LRFD [5.8.3.5]**.

The area of longitudinal reinforcement (in bottom of slab), A_s , should be checked for tensile capacity at the abutments, for dead load and (HL-93) live load on interior and exterior strips. The reinforcement at these locations shall have the capacity to resist the tension in the reinforcement produced by shear.

The factored shear, V_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. The factored tension force, T_{fact} , from shear, to be resisted is from **LRFD [Eq'n. 5.8.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = [\ V_u / \phi_v\ ] \cot\theta$$

Assume a diagonal crack would start at the inside edge of the bearing area. Assume the crack angle, θ, is 35 degrees. Calculate the distance from the bottom of slab to center of tensile reinforcement. Determine the distance D_{crack} from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement. Find the development length, ℓ_d , from Table 9.9-2, Chapter 9.

The nominal tensile resistance, T_{nom} , of the longitudinal bars at the crack location is:

$$T_{nom} = A_s f_y\ [\ D_{crack} - (\text{end cover})] / \ell_d \ \le\ A_s f_y$$

Then check that, $T_{fact} \le T_{nom}$ is satisfied.

If the values for T_{fact} and T_{nom} are close, the procedure for determining the crack angle, θ, as outlined in **LRFD [5.8.3.4.2]** should be used.

18.4.11 Uplift Check

Check for uplift at the abutments for (HL-93) live loads **LRFD [C3.4.1, 5.5.4.3]**. Compare the factored dead load reaction to the factored live load reaction. The reactions shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Place (HL-93) live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**.

18.4.12 Deflection Joints and Construction Joints

The designer should locate deflection joints in sidewalks and parapets on concrete slab structures according to the Standard *Vertical Face Parapet 'A'* in Chapter 30.

Refer to Standards *Continuous Haunched Slab* and *Continuous Flat Slab* in Chapter 18, for recommended construction joint guidelines.

18.4.13 Reinforcement Tables

Table 18.4-3 applies to: Rectangular Sections with Tension Reinforcement only

- Reinforcement Yield Strength (f_y) = 60,000 psi
- Concrete Compressive Strength (f'_c) = 4,000 psi

R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ
117.9	0.0020	335.6	0.0059	537.1	0.0098	722.6	0.0137	892.0	0.0176
123.7	0.0021	340.9	0.0060	542.1	0.0099	727.2	0.0138	896.1	0.0177
129.4	0.0022	346.3	0.0061	547.1	0.0100	731.7	0.0139	900.2	0.0178
135.2	0.0023	351.6	0.0062	552.0	0.0101	736.2	0.0140	904.4	0.0179
141.0	0.0024	357.0	0.0063	556.9	0.0102	740.7	0.0141	908.5	0.0180
146.7	0.0025	362.3	0.0064	561.8	0.0103	745.2	0.0142	912.5	0.0181
152.4	0.0026	367.6	0.0065	566.7	0.0104	749.7	0.0143	916.6	0.0182
158.1	0.0027	372.9	0.0066	571.6	0.0105	754.2	0.0144	920.7	0.0183
163.8	0.0028	378.2	0.0067	576.5	0.0106	758.7	0.0145	924.8	0.0184
169.5	0.0029	383.5	0.0068	581.4	0.0107	763.1	0.0146	928.8	0.0185
175.2	0.0030	388.8	0.0069	586.2	0.0108	767.6	0.0147	932.8	0.0186
180.9	0.0031	394.1	0.0070	591.1	0.0109	772.0	0.0148	936.9	0.0187
186.6	0.0032	399.3	0.0071	595.9	0.0110	776.5	0.0149	940.9	0.0188
192.2	0.0033	404.6	0.0072	600.8	0.0111	780.9	0.0150	944.9	0.0189
197.9	0.0034	409.8	0.0073	605.6	0.0112	785.3	0.0151	948.9	0.0190
203.5	0.0035	415.0	0.0074	610.4	0.0113	789.7	0.0152	952.9	0.0191
209.1	0.0036	420.2	0.0075	615.2	0.0114	794.1	0.0153	956.8	0.0192
214.8	0.0037	425.4	0.0076	620.0	0.0115	798.4	0.0154	960.8	0.0193
220.4	0.0038	430.6	0.0077	624.8	0.0116	802.8	0.0155	964.7	0.0194
225.9	0.0039	435.8	0.0078	629.5	0.0117	807.2	0.0156	968.7	0.0195
231.5	0.0040	441.0	0.0079	634.3	0.0118	811.5	0.0157	972.6	0.0196
237.1	0.0041	446.1	0.0080	639.0	0.0119	815.8	0.0158	976.5	0.0197
242.7	0.0042	451.3	0.0081	643.8	0.0120	820.1	0.0159	980.4	0.0198
248.2	0.0043	456.4	0.0082	648.5	0.0121	824.5	0.0160	984.3	0.0199
253.7	0.0044	461.5	0.0083	653.2	0.0122	828.8	0.0161	988.2	0.0200
259.3	0.0045	466.6	0.0084	657.9	0.0123	833.1	0.0162	992.1	0.0201
264.8	0.0046	471.7	0.0085	662.6	0.0124	837.3	0.0163	996.0	0.0202
270.3	0.0047	476.8	0.0086	667.3	0.0125	841.6	0.0164	999.8	0.0203
275.8	0.0048	481.9	0.0087	671.9	0.0126	845.9	0.0165	1003.7	0.0204
281.3	0.0049	487.0	0.0088	676.6	0.0127	850.1	0.0166	1007.5	0.0205
286.8	0.0050	492.1	0.0089	681.3	0.0128	854.3	0.0167	1011.3	0.0206
292.2	0.0051	497.1	0.0090	685.9	0.0129	858.6	0.0168	1015.1	0.0207
297.7	0.0052	502.2	0.0091	690.5	0.0130	862.8	0.0169	1018.9	0.0208
303.1	0.0053	507.2	0.0092	695.1	0.0131	867.0	0.0170	1022.7	0.0209
308.6	0.0054	512.2	0.0093	699.7	0.0132	871.2	0.0171	1026.5	0.0210
314.0	0.0055	517.2	0.0094	704.3	0.0133	875.4	0.0172	1030.3	0.0211
319.4	0.0056	522.2	0.0095	708.9	0.0134	879.5	0.0173	1034.0	0.0212
324.8	0.0057	527.2	0.0096	713.5	0.0135	883.7	0.0174	1037.8	0.0213
330.2	0.0058	532.2	0.0097	718.1	0.0136	887.9	0.0175	----	----

Table 18.4-3
R_u (psi) vs. ρ

R_u = coefficient of resistance (psi) = $M_u / \phi\, b\, d_s^2$ ρ = reinforcement ratio = $A_s / b\, d_s$

Table 18.4-4 can be used to select bar size and bar spacing to provide an adequate area of reinforcement to meet design requirements.

Bar Size Number	Nominal Dia. Inches	4 1/2"	5"	5 1/2"	6"	6 1/2"	7"	7 1/2"	8"	8 1/2"	9"	10"	12"
4	0.500	0.52	0.47	0.43	0.39	0.36	0.34	0.31	0.29	0.28	0.26	0.24	0.20
5	0.625	0.82	0.74	0.67	0.61	0.57	0.53	0.49	0.46	0.43	0.41	0.37	0.31
6	0.750	1.18	1.06	0.96	0.88	0.82	0.76	0.71	0.66	0.62	0.59	0.53	0.44
7	0.875	1.60	1.44	1.31	1.20	1.11	1.03	0.96	0.90	0.85	0.80	0.72	0.60
8	1.000	2.09	1.88	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	0.94	0.79
9	1.128	--	2.40	2.18	2.00	1.85	1.71	1.60	1.50	1.41	1.33	1.20	1.00
10	1.270	--	3.04	2.76	2.53	2.34	2.17	2.02	1.90	1.79	1.69	1.52	1.27
11	1.410	--	3.75	3.41	3.12	2.88	2.68	2.50	2.34	2.21	2.08	1.87	1.56

Table 18.4-4
Area of Bar Reinf. (in^2 / ft) vs. Spacing of Bars (in)

18.5 Design Example

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E18-1 Continuous 3-Span Haunched Slab - LRFD

A continuous 3-span haunched slab structure is used for the design example. The same basic procedure is applicable to continuous flat slabs. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. Design using a slab width equal to one foot. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

E18-1.1 Structure Preliminary Data

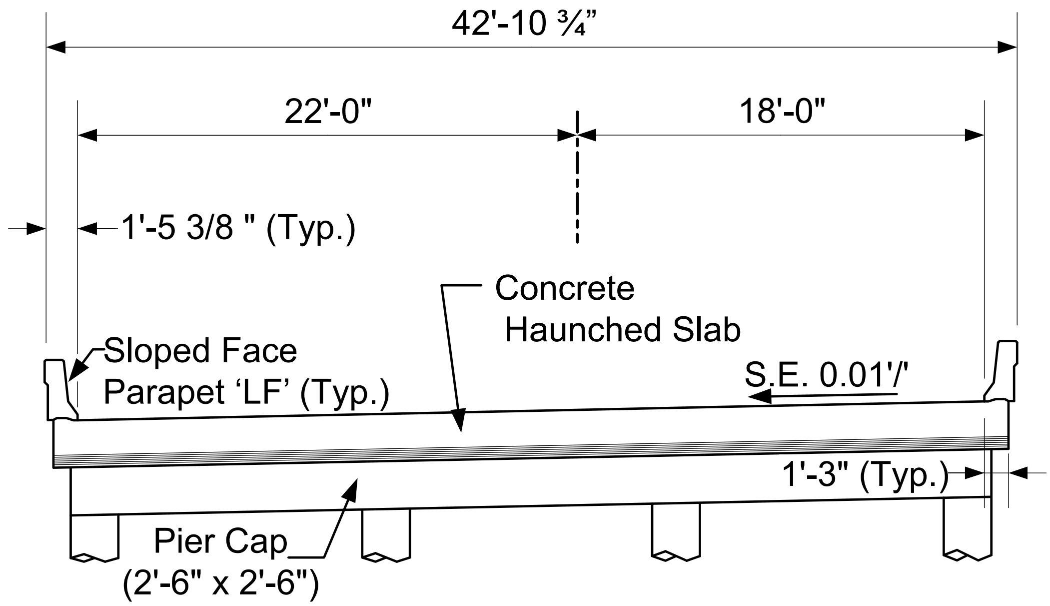

Figure E18.1

Section Perpendicular to Centerline

Live Load: HL-93
(A1) Fixed Abutments at both ends
Parapets placed after falsework is released

Geometry:

$L_1 := 38.0$	ft	Span 1
$L_2 := 51.0$	ft	Span 2
$L_3 := 38.0$	ft	Span 3
$slab_{width} := 42.5$	ft	out to out width of slab
$skew := 6$	deg	skew angle (RHF)
$w_{roadway} := 40.0$	ft	clear roadway width

Material Properties: (See 18.2.2)

$f'_c := 4$	ksi	concrete compressive strength

$f_y := 60$ ksi yield strength of reinforcement

$E_c := 3800$ ksi modulus of elasticity of concrete

$E_s := 29000$ ksi modulus of elasticity of reinforcement

$n := 8$ E_s / E_c (modular ratio)

Weights:

$w_c := 150$ pcf concrete unit weight

$w_{LF} := 387$ plf weight of Type LF parapet (each)

E18-1.2 LRFD Requirements

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: (See 18.3.2.1)

$$Q = \Sigma\eta_i \cdot \gamma_i \cdot Q_i \le \phi \cdot R_n = R_r \qquad \text{(Limit States Equation)}$$

The value of the load modifier is:

$\eta_i := 1.0$ for all Limit States (See 18.3.2.2)

The force effect, Q_i, is the moment, shear, stress range or deformation caused by applied loads.

The applied loads from **LRFD [3.3.2]** are:

DC = dead load of slab (DC_{slab}), ½ inch wearing surface ($DC_{1/2"WS}$) and parapet dead load (DC_{para}) - (See E18-1.3)

DW = dead load of future wearing surface (DW_{FWS}) - (See E18-1.3)

LL+IM = vehicular live load (LL) with dynamic load allowance (IM) - (See E18-1.4)

The Influence of ADTT and skew on force effects, Q_i, are ignored for slab bridges (See 18.3.2.2).

The values for the load factors, γ_i, (for each applied load) and the resistance factors, ϕ, are found in Table E18.1.

The total factored force effect, Q, must not exceed the factored resistance, R_r. The nominal resistance, R_n, is the resistance of a component to the force effects.

		Strength I	Service I	Fatigue I
Load Factor (DC)	γ_{DC}	LRFD Table 3.4.1-2 0.90 (min.) 1.25 (max.)	LRFD Table 3.4.1-1 1.00	---
Load Factor (DW)	γ_{DW}	LRFD Table 3.4.1-2 0.65 (min.) 1.50 (max.)	LRFD Table 3.4.1-1 1.00	---
Load Factor (LL+IM)	γ_{LL+IM}	LRFD Table 3.4.1-1 1.75	LRFD Table 3.4.1-1 1.00	LRFD Table 3.4.1-1 1.50
Resistance Factor	ϕ	LRFD 5.5.4.2 0.90 flexure[1] 0.90 shear	LRFD 1.3.2.1 1.00	LRFD C1.3.2.1 1.00

Table E18.1
Load and Resistance Factors

1 All reinforced concrete sections in this example were found to be tension-controlled sections as defined in **LRFD [5.7.2.1]**; therefore $\phi_f = 0.90$

E18-1.3 Trial Slab Depth and Dead Loads (DC, DW)

Refer to Table 18.2-1 in 18.2.3 for an interior span length, L_2 , of 51 feet. The trial slab depth, d_{slab} (not including the 1/2 inch wearing surface), is estimated at:

$d_{slab} := 17$ in

The haunch depth, D_{haunch} , is approximately equal to d_{slab} divided by 0.6:

$$D_{haunch} := \frac{d_{slab}}{0.6} \rightarrow \frac{17}{0.6}$$

$D_{haunch} := \text{round}(D_{haunch})$ $\boxed{D_{haunch} = 28}$ in

D_{haunch} does not include the 1/2 inch wearing surface.

The length of the haunch, L_{haunch} , measured from the C/L of pier to the end of haunch, is approximately (0.15 to 0.18)*L_2. (L_2 equals interior span length = 51 feet)

$L_{haunchMin} := 0.15 \cdot L_2$ $\boxed{L_{haunchMin} = 7.65}$ ft

$L_{haunchMax} := 0.18 \cdot L_2$ $\boxed{L_{haunchMax} = 9.18}$ ft

Select the value for L_{haunch} to the nearest foot : $\boxed{L_{haunch} = 8}$ ft

The slab dead load, DC_{slab} , and the section properties of the slab, do not include the 1/2 inch wearing surface.

The dead load for the 17 inch slab depth, for a one foot design width, is calculated as follows:

$DC_{17slab} := \frac{d_{slab}}{12} \cdot 1.0 \cdot w_c \rightarrow \frac{17}{12} \cdot 1.0 \cdot 150$ $\boxed{DC_{17slab} = 213}$ plf

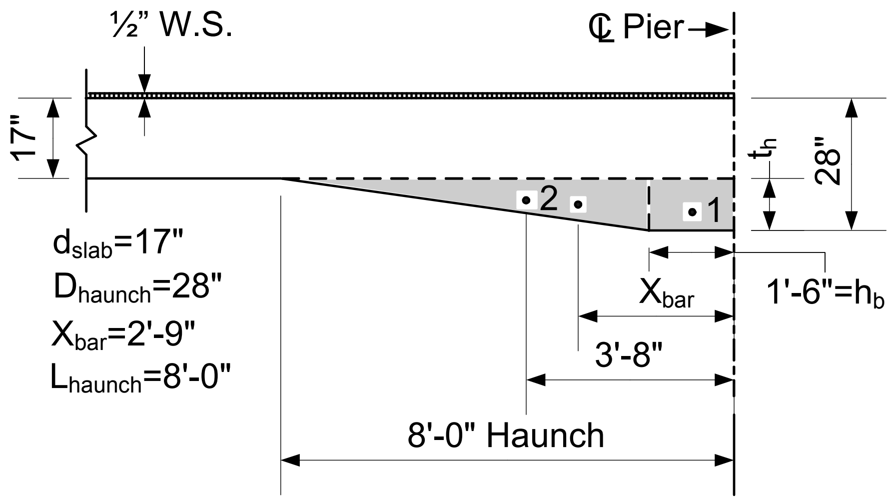

Figure E18.2

Haunched Section at Pier

For hand computations, determine the partial haunch dead load in the shaded area in Figure E18.2. Determine the center of gravity, X_{bar}, for this area and distribute its weight uniformly over twice this distance. Haunch dead load is often computed by computer programs.

The partial haunch thickness, t_h, equals:

$t_h := D_{haunch} - d_{slab}$ $\boxed{t_h = 11}$ in

For a 2.5 ft. wide pier cap, the bottom width of the haunch is: $h_b := \frac{2.5}{2} + 0.25$ $\boxed{h_b = 1.5}$ ft

The area of sections (1 & 2) in Figure E18.2 and the location of their center of gravity is:

$A_1 := h_b \cdot \frac{t_h}{12}$ $A_2 := \frac{(L_{haunch} - h_b) \cdot \frac{t_h}{12}}{2}$ $\boxed{A_1 = 1.38}$ ft^2 $\boxed{A_2 = 2.98}$ ft^2

$X_{bar1} := \frac{h_b}{2}$ $X_{bar2} := \frac{L_{haunch} - h_b}{3} + h_b$ $\boxed{X_{bar1} = 0.75}$ ft $\boxed{X_{bar2} = 3.67}$ ft

The location of the center of gravity, X_{bar}, of the shaded area in Figure E18.2 is:

$$X_{bar} := \frac{A_1 \cdot X_{bar1} + A_2 \cdot X_{bar2}}{A_1 + A_2} \qquad \boxed{X_{bar} = 2.75} \text{ ft}$$

The haunch dead load is uniformly distributed over a distance of $2 \cdot X_{bar} = 5.5$ feet. For a one foot design width, its value is calculated as follows:

$$DC_{haunch} := \frac{A_1 + A_2}{2 \cdot X_{bar}} \cdot 1.0 \cdot w_c \qquad \boxed{DC_{haunch} = 119} \text{ plf}$$

The dead load of the slab, DC_{slab} , is the total dead load from DC_{17slab} and DC_{haunch}.

The parapet dead load is uniformly distributed over the full width of the slab when designing for an interior strip of slab. The parapet dead load on a one foot design width, for an interior strip, is calculated as follows:

$$DC_{para} := \frac{2 \cdot w_{LF}}{slab_{width}} \rightarrow \frac{2 \cdot 387}{42.5} \qquad \boxed{DC_{para} = 18} \text{ plf}$$

The parapet dead load is uniformly distributed over the exterior strip width of the slab when designing for an exterior strip (edge beam).

The 1/2 inch wearing surface dead load and a possible future wearing surface (FWS) dead load must also be included in the design of the slab. Therefore for a one foot design width:

$$DC_{1/2"WS} = (0.5/12)(1.0)(w_c) \qquad \boxed{DC_{1/2"WS} = 6} \text{ plf}$$

$$DW_{FWS} = (20)(1.0) \qquad \boxed{DW_{FWS} = 20} \text{ plf}$$

E18-1.4 Vehicular Live Load (LL) and Dynamic Load Allowance (IM)

The live load combinations used for design are:

LL#1:	Design Tandem (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#2:	Design Truck (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#3:	90% [Double Design Trucks (+ IM) + Design Lane Load]	**LRFD [3.6.1.3.1]**
LL#4:	Fatigue Truck (+ IM)	**LRFD [3.6.1.4.1]**
LL#5:	Design Truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6:	25% [Design Truck (+ IM)] + Design Lane Load	**LRFD [3.6.1.3.2]**

Table E18.2
Live Load Combinations

The live load combinations and dynamic load allowance, IM, **LRFD [3.6.2]** are applied to the Limit States as shown in Table E18.3.

Where (IM) is required, multiply the loads by (1 + IM/100) to include the dynamic effects of the load. (IM) is not applied to the Design Lane Load.

The live load force effect, Q_i , shall be taken as the largest from the live loads shown in Table E18.3 for that Limit State.

Strength I Limit State:	LL#1 , LL#2 , LL#3 [1]	IM = 33%
Service I Limit State: (for crack control criteria)	LL#1 , LL#2 , LL#3 [1]	IM = 33%
Service I Limit State: (for LL deflection criteria)	LL#5 , LL#6	IM = 33%
Fatigue I Limit State:	LL#4 (single Fatigue Truck)	IM = 15%

Table E18.3
Live Loads for Limit States

[1] (LL#3) is used to calculate negative live load moments between points of contraflexure and also reactions at interior supports. The points of contraflexure are located by placing a uniform load across the entire structure. For these moments and reactions, the results calculated from (LL#3) are compared with (LL#1) and (LL#2) results, and the critical value is selected.

E18-1.5 Minimum Slab Thickness Criteria

Check adequacy of chosen slab thickness by looking at live load deflection and dead load deflection (camber) criteria, using Service I Limit State.

E18-1.5.1 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2]**, using Service I Limit State.

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLser1} := 1.0$ $\phi_{ser1} := 1.0$

$Q_i = \Delta_{LLser1}$ = largest live load deflection caused by live loads (LL#5 or LL#6)

See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM)

$$Q = \eta_i \cdot \gamma_{LLser1} \cdot \Delta_{LLser1} = (1.0) \cdot (1.0) \cdot \Delta_{LLser1}$$

Use (3) design lanes **LRFD [3.6.1.1.1]**, multiple presence of live load factor (m=0.85) **LRFD [3.6.1.1.2]** and gross moment of Inertia, I_g , based on the entire slab width acting as a unit, to calculate live load deflection. Use modulus of elasticity, E_c = 3800 ksi.

$R_n = \frac{L}{1200}$ = max. live load def'l. (L = span length)

$R_r = \phi_{ser1} \cdot R_n = 1.00 \cdot \frac{L}{1200}$

Therefore: $\Delta_{LLser1} \le \frac{L}{1200}$ (Limit States Equation)

The largest live load deflection is caused by live load (LL#5)

Span 1: $\Delta_{LLser1} = 0.29$ in $< \frac{L_1}{1200} = 0.38$ in <u>O.K.</u>

Span 2: $\Delta_{LLser1} = 0.47$ in $< \frac{L_2}{1200} = 0.51$ in <u>O.K.</u>

E18-1.5.2 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits **LRFD [5.7.3.6.2]**, using Service I Limit State. Dead load deflections are computed using the gross moment of inertia, I_g. All dead loads are to be uniformly distributed across the slab width.

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCser1} := 1.0$ $\gamma_{DWser1} := 1.0$ $\phi_{ser1} := 1.0$

$Q_i = \Delta_{DL}$ = dead load deflection due to <u>applied loads</u> (DC, DW) as stated in E18-1.2.

$Q = \eta_i \cdot \gamma \cdot (\Delta_{DL}) = (1.0) \cdot (1.0) \cdot (\Delta_{DL})$

The Bureau of Structures currently calculates full camber as three times the dead load deflection. The maximum allowable camber for continuous spans is 1 3/4 inches (See 18.4.4.2). Therefore, the allowable dead load deflection is 1/3 of the maximum allowable camber. Use modulus of elasticity, E_c = 3800 ksi.

R_n = (max. allowable camber)/3 = 1 3/4 inches /3 = 0.583 inches

$R_r = \phi_{ser1} \cdot R_n = 1.00 \cdot (0.583)$ = 0.583 in

Therefore: $\Delta_{DL} \le 0.583$ in (Limit States Equation)

Δ_{DL} (at 0.4 pt Span 1) = 0.17 in < 0.583 in <u>O.K.</u>

Δ_{DL} (at C/L of Span 2) = 0.27 in < 0.583 in <u>O.K.</u>

E18-1.6 Live Load Distribution (Interior Strip)

Live loads are distributed over an equivalent width, E, as calculated below. Equivalent strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3].** The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load. The equivalent distribution width applies for both live load moment and shear.

Single - Lane Loading: $E = 10.0 + 5.0 \cdot (L_1 \cdot W_1)^{0.5}$

Multi - Lane Loading: $E = 84.0 + 1.44 \cdot (L_1 \cdot W_1)^{0.5} \leq 12.0 \cdot \frac{W}{N_L}$

Where:

E = equivalent distribution width (in)

L_1 = modified span length taken equal to the lesser of the actual span or 60.0 ft (ft)

W_1 = modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60.0 ft for multi-lane loading, or 30.0 ft for single-lane loading (ft)

W = physical edge to edge width of bridge (ft)

N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

For single-lane loading:

(Span 1, 3) $E := 10.0 + 5.0 \cdot (38 \cdot 30)^{0.5}$ $\boxed{E = 178}$ in

(Span 2) $E := 10.0 + 5.0 \cdot (51 \cdot 30)^{0.5}$ $\boxed{E = 205}$ in

For multi-lane loading:

$$12.0 \cdot \frac{W}{N_L} = 12.0 \cdot \frac{42.5}{3} = 170 \text{ in}$$

(Span 1, 3) $E := 84.0 + 1.44 \cdot (38 \cdot 42.5)^{0.5}$ $\boxed{E = 141}$ in < 170 in O.K.

(Span 2) $E := 84.0 + 1.44 \cdot (51 \cdot 42.5)^{0.5}$ $\boxed{E = 151}$ in < 170 in O.K.

E18-1.6.1 Strength and Service Limit State

Use the smaller equivalent widths, which are from multi-lane loading, when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The distribution factor (DF) is computed for a design slab width equal to one foot.

$DF = \frac{1}{E}$ (where E is in feet)

The multiple presence factor (m) has been included in the equations for distribution width (E) and therefore aren't used to adjust the distribution factor (DF) **LRFD [3.6.1.1.2]**.

For spans 1 & 3: (E = 141" = 11.75')

$$DF := \frac{1}{11.75} \qquad \boxed{DF = 0.0851} \ \frac{lanes}{ft - slab}$$

For span 2: (E =151" = 12.583')

$$DF := \frac{1}{12.583} \qquad \boxed{DF = 0.0795} \ \frac{lanes}{ft - slab}$$

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore, use DF = 0.0851 lanes/ft-slab for all spans.

E18-1.6.2 Fatigue Limit State

Use equivalent widths from single-lane loading to check fatigue stress range criteria. For the Fatigue Limit State only one design truck (Fatigue Truck) is present **LRFD [3.6.1.4]**. Calculate the distribution factor (DF) and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **LRFD [3.6.1.1.2]**.

The distribution factor (DF) is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E \cdot (1.20)} \qquad \text{(where E is in ft)}$$

For spans 1 & 3: (E = 178" = 14.833')

$$DF := \frac{1}{(14.833)\cdot(1.20)} \qquad \boxed{DF = 0.0562} \ \frac{lanes}{ft - slab}$$

For span 2: (E =205" = 17.083')

$$DF := \frac{1}{(17.083)\cdot(1.20)} \qquad \boxed{DF = 0.0488} \ \frac{lanes}{ft - slab}$$

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore, use DF = 0.0562 lanes/ft.-slab for all spans.

Table E18.4 Unfactored Moments (kip - ft) (on a one foot design width) **Interior Strip**

Point	M_{DC} [1]	M_{DW} [2]	DF=0.0851 (IM not used) +Design Lane	DF=0.0851 (IM not used) -Design Lane	DF=0.0851 (incl. IM=33%) +Design Tandem	DF=0.0851 (incl. IM=33%) -Design Tandem
0.1	9.6	0.8	3.2	-1.0	17.2	-3.2
0.2	15.9	1.3	5.5	-1.9	29.0	-6.4
0.3	18.7	1.6	7.1	-2.9	35.5	-9.6
0.4	18.1	1.5	7.9	-3.8	37.5	-12.8
0.5	14.1	1.2	7.9	-4.8	36.2	-16.0
0.6	6.6	0.6	7.2	-5.7	31.9	-19.2
0.7	-4.2	-0.4	5.6	-6.6	24.7	-22.3
0.789	-17.1	-1.5	3.7	-7.6	16.8	-25.1
0.8	-18.5	-1.6	3.5	-7.8	15.8	-25.5
0.9	-36.5	-3.1	2.4	-10.8	8.4	-28.7
1.0	-59.2	-4.9	2.2	-15.5	9.2	-31.9
1.1	-29.8	-2.5	1.9	-8.8	7.6	-21.8
1.157	-16.9	-1.4	2.3	-6.2	13.8	-19.8
1.2	-8.1	-0.7	2.9	-4.9	18.9	-18.4
1.3	7.2	0.6	5.4	-3.8	28.9	-14.9
1.4	16.4	1.4	7.5	-3.8	35.4	-11.4
1.5	19.6	1.6	8.2	-3.8	37.4	-8.0

Point	DF=0.0851 (incl. IM=33%) +Design Truck	DF=0.0851 (incl. IM=33%) -Design Truck	DF=0.0851 [3] (IM not used) (90%) of -Design Lane	DF=0.0851 [3] (incl. IM=33%) (90%) of -Double Design Trucks	DF=0.0562 (incl. IM=15%) +Fatigue Truck	DF=0.0562 (incl. IM=15%) -Fatigue Truck
0.1	18.1	-3.9	---	---	7.7	-1.4
0.2	29.3	-7.7	---	---	12.9	-2.8
0.3	34.4	-11.6	---	---	15.8	-4.2
0.4	35.4	-15.4	---	---	16.7	-5.5
0.5	33.9	-19.3	---	---	16.0	-6.9
0.6	30.7	-23.1	---	---	14.3	-8.4
0.7	23.3	-27.0	-6.0	-24.3	11.3	-9.8
0.789	14.0	-30.5	-6.9	-27.4	7.8	-11.0
0.8	13.0	-30.9	-7.0	-27.8	7.5	-11.2
0.9	9.0	-34.7	-9.7	-31.4	3.9	-16.0
1.0	10.1	-39.9	-13.9	-35.0	3.9	-23.0
1.1	8.0	-23.8	-8.0	-22.6	4.6	-13.6
1.157	12.1	-21.7	-5.6	-20.2	6.9	-9.0
1.2	15.3	-20.1	-4.4	-18.5	8.7	-7.7
1.3	27.7	-16.4	---	---	13.1	-6.3
1.4	35.4	-12.5	---	---	15.9	-4.8
1.5	37.2	-8.8	---	---	16.7	-3.4

Superscripts for Table E18.4 are defined on the following page.

In Table E18.4:

[1] M_{DC} is moment due to slab dead load (DC_{slab}), parapet dead load (DC_{para}) after its weight is distributed across width of slab, and 1/2 inch wearing surface ($DC_{1/2"WS}$).

[2] M_{DW} is moment due to future wearing surface (DW_{FWS}).

[3] The points of contraflexure are located at the (0.66 pt.) of span 1 and the (0.25 pt.) of span 2, when a uniform load is placed across the entire structure. Negative moments in these columns are shown between the points of contraflexure per **LRFD [3.6.1.3.1]**.

E18-1.7 Longitudinal Slab Reinforcement (Interior Strip)

Select longitudinal reinforcement for an Interior Strip.

The concrete cover on the top bars is 2 1/2 inches, which includes a 1/2 inch wearing surface. The bottom bar cover is 1 1/2 inches. (See 18.4.6)

E18-1.7.1 Positive Moment Reinforcement for Span 1

Examine the 0.4 point of span 1

E18-1.7.1.1 Design for Strength

Design reinforcement using Strength I Limit State and considerations and assumptions detailed in **LRFD [5.5.4, 5.7.2]**

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCmax} := 1.25$ $\gamma_{DWmax} := 1.50$ $\gamma_{LLstr1} := 1.75$ $\phi_f := 0.9$

$Q_i = M_{DC}, M_{DW}, M_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; moments due to applied loads as stated in E18-1.2

$$Q = M_u = \eta_i \left[\gamma_{DCmax} (M_{DC}) + \gamma_{DWmax} (M_{DW}) + \gamma_{LLstr1} (M_{LL+IM}) \right]$$

$$= 1.0 \left[1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \right]$$

$$R_n = M_n = A_s \cdot f_s \cdot \left(d_s - \frac{a}{2} \right) \qquad \text{(See 18.3.3.2.1)}$$

$$M_r = \phi_f \cdot M_n = 0.90 \cdot A_s \cdot f_s \cdot \left(d_s - \frac{a}{2} \right)$$

Therefore : $M_u \le M_r$ (Limit States Equation)

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \le 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM)

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.4 pt. - span 1):

M_{DC} = 18.1 kip-ft M_{DW} = 1.5 kip-ft M_{LL+IM} = 7.9 + 37.5 = 45.4 kip-ft

$M_u := 1.25\cdot(18.1) + 1.50\cdot(1.5) + 1.75\cdot(45.4)$ $\boxed{M_u = 104.3}$ kip-ft

$b := 12$ inches (for a one foot design width)

$d_s = d_{slab}$ - bott. bar clr. - 1/2 bott. bar dia.

$d_s := 17 - 1.5 - 0.6$ $\boxed{d_s = 14.9}$ in

Calculate R_u, coefficient of resistance:

$R_u = \dfrac{M_u}{\phi_f\cdot b\cdot d_s^{\,2}}$ $R_u := \dfrac{104.3\cdot(12)\cdot 1000}{0.9\cdot(12)\cdot 14.9^2}$ $\boxed{R_u = 522}$ psi

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.0095$

$A_s = \rho\cdot(b)\cdot d_s$ $A_s := 0.0095\cdot(12)14.9$ $\boxed{A_s = 1.7}$ $\dfrac{in^2}{ft}$

Try: #9 at 7" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

Calculate the depth of the compressive stress block.

Assume $f_s = f_y$ (See 18.3.3.2.1) ; for f'_c = 4.0 ksi : $\alpha_1 := 0.85$ and $\beta_1 = 0.85$

$a = \dfrac{A_s\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $a := \dfrac{1.71\cdot(60)}{0.85\cdot(4.0)\cdot 12}$ $\boxed{a = 2.51}$ in

If $\dfrac{c}{d_s} \le 0.6$ for (f_y= 60 ksi) **LRFD [5.7.2.1]**, then reinforcement has yielded and the assumption is correct.

$\beta_1 := 0.85$ $c := \dfrac{a}{\beta_1}$ $\boxed{c = 2.96}$ in

$\dfrac{c}{d_s} = 0.2 < 0.6$ therefore, the reinforcement will yield.

$M_r = 0.90\cdot A_s\cdot f_y\cdot\left(d_s - \dfrac{a}{2}\right)$

$M_r := 0.9\cdot(1.71)\cdot 60.0\cdot\left(\dfrac{14.9 - \dfrac{2.51}{2}}{12}\right)$ $\boxed{M_r = 105}$ kip-ft

Therefore, M_u = 104.3 kip-ft < M_r = 105 kip-ft O.K.

E18-1.7.1.2 Check for Fatigue

Check reinforcement using Fatigue I Limit State and considerations and assumptions detailed in **LRFD [5.5.3, 5.7.1, 9.5.3]**.

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLfatigue} := 1.5$ $\phi_{fatigue} := 1.0$

When reinforcement remains in tension throughout the fatigue cycle,

$Q_i = \Delta f = f_{range}$ = stress range in bar reinforcement due to flexural moment range (M_{range}) caused by Fatigue Truck (LL#4). See Table E18.2 and E18.3 in E18-1.4 for description of live load and dynamic load allowance (IM)

$$Q = \eta_i \cdot \gamma_{LLfatigue} \cdot f_{range} = (1.0)\cdot(1.5)\cdot f_{range}$$

$$R_n = (\Delta F_{TH}) = 24 - 0.33 \cdot f_{min}$$ for f_y = 60 ksi (See 18.3.5.2.1)

$$R_r = \phi_{fatigue} \cdot R_n = 1.0 \cdot (24 - 0.33 \cdot f_{min})$$

Therefore: $1.5 \cdot (f_{range}) \le 24 - 0.33 \cdot f_{min}$ (Limit States Equation)

From Table E18.4, the moments at (0.4 pt.) of span 1 are:

M_{DC} = 18.1 kip-ft M_{DW} = 1.5 kip-ft

+Fatigue Truck = 16.7 kip-ft -Fatigue Truck = -5.5 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

The section properties for fatigue shall be based on cracked sections where the sum of stresses, due to permanent loads, and ($\gamma_{LLfatigue}$ = 1.5) times the fatigue load is tensile and exceeds $0.095\sqrt{f'_c}$ **LRFD [5.5.3.1]**

Allowable tensile stress for fatigue (cracking stress):

$$f_{tensile} = 0.095\sqrt{f'_c} = 0.095 \cdot \sqrt{4}$$ $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) +1.5(Fatigue Truck)

$M_{fatigueMax} := 1.0 \cdot (18.1) + 1.0(1.5) + 1.5(16.7)$ $\boxed{M_{fatigueMax} = 44.65}$ kip-ft (tension)

$M_{fatigueMin} := 1.0 \cdot (18.1) + 1.0(1.5) + 1.5(-5.5)$ $\boxed{M_{fatigueMin} = 11.35}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$: $f_{fatigue} = \frac{M_{fatigue} \cdot (y)}{I_g}$

$y = \frac{d_{slab}}{2} = \frac{17}{2}$ $\boxed{y = 8.5}$ in

$I_g = \frac{1}{12} \cdot b \cdot d_{slab}^3 = \frac{1}{12} \cdot (12)\, 17^3$ $\boxed{I_g = 4913}$ in^4

$f_{fatigueMax} := \frac{M_{fatigueMax} \cdot (y) \cdot 12}{I_g}$ $\boxed{f_{fatigueMax} = 0.93}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$f_{fatigueMin} := \frac{M_{fatigueMin} \cdot (y) \cdot 12}{I_g}$ $\boxed{f_{fatigueMin} = 0.24}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (+ Fatigue Truck) - (-Fatigue Truck)

$M_{range} := 16.7 - (-5.5)$ $\boxed{M_{range} = 22.2}$ kip-ft

The moment arm used in equations below is: (j) (d_s) Therefore, using:

$A_s = 1.7 \frac{in^2}{ft}$ (required for strength), $d_s = 14.9$ in , $n := 8$, and transformed section analysis, gives a value of $j := 0.893$

$f_{range} = \frac{M_{range}}{A_s \cdot (j) \cdot d_s} = \frac{22.2 \cdot 12}{1.7 \cdot (0.893)\, 14.9}$ $\boxed{f_{range} = 11.78}$ ksi

$f_{range1.5} := 1.5 \cdot f_{range}$ $\boxed{f_{range1.5} = 17.67}$ ksi

$$f_{min} = \frac{M_{DC} + M_{DW} + 1.5(-FatigueTruck)}{A_s \cdot (j) \cdot d_s}$$

$f_{min} := \frac{[18.1 + 1.5 + 1.5 \cdot (-5.5)] \cdot 12}{1.7 \cdot (0.893)\, 14.9}$ $\boxed{f_{min} = 6.02}$ ksi

$R_r := 24 - 0.33 \cdot f_{min}$ $\boxed{R_r = 22.01}$ ksi

Therefore, $1.5 \cdot (f_{range}) = 17.67$ ksi < $R_r = 22.01$ ksi   O.K.

E18-1.7.1.3 Check Crack Control

Check reinforcement using Service I Limit State and considerations and assumptions detailed in **LRFD [5.5.2, 5.7.1, 5.7.3.4]**

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**; λ = 1.0 (normal wgt. conc.)**LRFD [5.4.2.8]**

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ $\quad \boxed{f_r = 0.48}$ ksi $\quad f_{r80\%} := 0.8 \cdot f_r$ $\quad \boxed{f_{r80\%} = 0.38}$ ksi

$$f_T = \frac{M_s \cdot (c)}{I_g}$$

$c := \frac{d_{slab}}{2}$ $\quad \boxed{c = 8.5}$ in $\quad I_g := \frac{1}{12} \cdot b \cdot d_{slab}^{\ 3}$ $\quad \boxed{I_g = 4913}$ in^4

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DC.ser1} := 1.0$ $\quad \gamma_{DW.ser1} := 1.0$ $\quad \gamma_{LLser1} := 1.0$ $\quad \phi_{ser1} := 1.0$

$Q_i = M_{DC}, M_{DW}, M_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; moments due to applied loads as stated in E18-1.2

$Q = M_S = \eta_i\,[\gamma_{DC.ser1}(M_{DC}) + \gamma_{DW.ser1}(M_{DW}) + \gamma_{LLser1}(M_{LL+IM})]$

$= 1.0\,[1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})]$

Therefore, M_s becomes:

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$ (Factored Load Equation)

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.1.1, at (0.4 pt.) of span 1, provides:

M_{DC} = 18.1 kip-ft $\quad M_{DW}$ = 1.5 kip-ft $\quad M_{LL+IM}$ = 7.9 + 37.5 = 45.4 kip-ft (LL#1)

$M_s := 1.0 \cdot (18.1) + 1.0 \cdot (1.5) + 1.0 \cdot (45.4)$ $\quad \boxed{M_s = 65}$ kip-ft

$f_T = \frac{M_s \cdot (c)}{I_g}$ $\quad f_T := \frac{65.0 \cdot (8.5) \cdot 12}{4913}$ $\quad \boxed{f_T = 1.35}$ ksi

f_T = 1.35 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

Knowing $A_s = 1.7 \frac{in^2}{ft}$ (required for strength)

Try: #9 at 7" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

The spacing (s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot (d_c) \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

d_c = clr. cover + 1/2 bar dia.
= thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in). See Figure E18.3

$$d_c := 1.5 + \frac{1.128}{2} \qquad \boxed{d_c = 2.064} \text{ in}$$

h = overall depth of the section (in). See Figure E18.3

$$h := d_{slab} \qquad \boxed{h = 17} \text{ in}$$

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.2}$$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\le 0.6\ f_y$

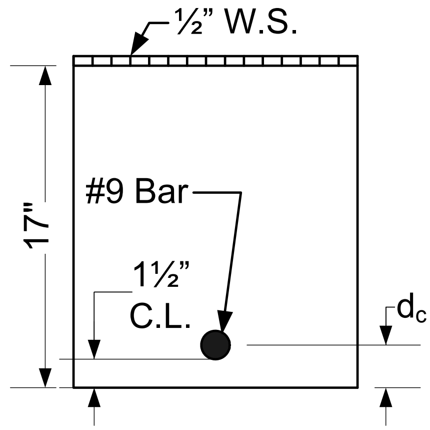

Figure E18.3

Cross Section - (0.4 pt.) Span 1

The moment arm used in the equation below to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.1.2, j = 0.893

$$f_{ss} = \frac{M_s}{A_s \cdot (j) \cdot (h - d_c)} = \frac{65.0 \cdot (12)}{1.71 \cdot (0.893)(17 - 2.064)} \qquad \boxed{f_{ss} = 34.2} \text{ ksi} \quad \le 0.6\ f_y \text{ O.K.}$$

$$s \le \frac{700 \cdot (1.00)}{1.2 \cdot (34.2)} - 2 \cdot (2.064) = 17.0 - 4.1 = 12.9 \quad \text{in}$$

$s \le 12.9$ in

Therefore, spacing prov'd. = 7 in < 12.9 in O.K.

Use: #9 at 7" c-c spacing in span 1 (Max. positive reinforcement).

E18-1.7.1.4 Minimum Reinforcement Check

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD[5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S$ where: $S = \frac{I_g}{c}$ therefore, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$f_r = 0.24\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.48}$ ksi

$I_g := \frac{1}{12} \cdot b \cdot d_{slab}{}^3$ $\boxed{I_g = 4913}$ in^4 $c := \frac{d_{slab}}{2}$ $\boxed{c = 8.5}$ in

$M_{cr} = \frac{1.1f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (4913)}{8.5(12)}$ $\boxed{M_{cr} = 25.43}$ kip-ft

$1.33 \cdot M_u = 138.75$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.1.1 and (M_u = 104.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

As shown in E18-1.7.1.1, the reinforcement yields, therefore:

$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$ $\boxed{M_r = 105}$ kip-ft

Therefore, M_{cr} = 25.43 kip-ft < M_r = 105 kip-ft O.K.

E18-1.7.2 Negative Moment Reinforcement at Piers

Examine at C/L of Pier

E18-1.7.2.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The negative live load moment shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#2), therefore at (C/L of Pier):

M_{DC} = -59.2 kip-ft $\quad$ M_{DW} = -4.9 kip-ft $\quad$ M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft

$M_u := 1.25\cdot(-59.2) + 1.50\cdot(-4.9) + 1.75\cdot(-55.4)$ $\quad$ $\boxed{M_u = -178.3}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 25.4}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 307.1}$ psi $\quad$ $\boxed{\rho = 0.0054}$ $\quad$ $\boxed{A_s = 1.65}$ $\frac{in^2}{ft}$

Try: #8 at 5 1/2" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.51}$ in

Then, $\boxed{c = 2.96}$ in and $\frac{c}{d_s} = 0.12 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 186.6}$ kip-ft

Therefore, M_u = 178.3 kip-ft < M_r = 186.6 kip-ft O.K.

E18-1.7.2.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.5\cdot\left(f_{range}\right) \leq 24 - 0.33\cdot f_{min}$ $\quad$ (for f_y = 60 ksi)

From Table E18.4, the moments at (C/L Pier) are:

M_{DC} = -59.2 kip-ft $\quad$ M_{DW} = -4.9 kip-ft

+Fatigue Truck = 3.9 kip-ft -Fatigue Truck = -23.0 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

Allowable tensile stress for fatigue (cracking stress): $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(\text{Fatigue Truck})$

$\boxed{M_{fatigueMax} = -98.6}$ kip-ft (tension) $\boxed{M_{fatigueMin} = -58.3}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$, where : $\boxed{y = 14}$ in $\boxed{I_g = 21952}$ in^4

$$f_{fatigue} = \frac{M_{fatigue} \cdot (y)}{I_g}$$

$\boxed{f_{fatigueMax} = 0.75}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$\boxed{f_{fatigueMin} = 0.45}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (- Fatigue Truck) - (+Fatigue Truck) $\boxed{M_{range} = -26.9}$ kip-ft

The values for A_s, d_s, n and j (from transformed section) used to calculate f_{range} and f_{min} are:

$A_s = 1.65 \dfrac{in^2}{ft}$ (required for strength), $d_s = 25.4$ in , $n := 8$, $j := 0.915$

The values for f_{range}, $f_{range1.5}$, and f_{min} are:

$\boxed{f_{range} = 8.42}$ ksi $\boxed{f_{range1.5} = 12.63}$ ksi $\boxed{f_{min} = 18.23}$ ksi

The factored resistance is: $\boxed{R_r = 17.98}$ ksi

Therefore, $1.5 \cdot (f_{range})$ = 12.63 ksi < R_r = 17.98 ksi <u>O.K.</u>

E18-1.7.2.3 Check Crack Control

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 14}$ in $\boxed{I_g = 21952}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.2.1, at (C/L of Pier), provides:

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft (LL#2)

$M_s := 1.0\cdot(59.2) + 1.0\cdot(4.9) + 1.0\cdot(55.4)$ $\boxed{M_s = 119.5}$ kip-ft

$f_T = \dfrac{M_s \cdot c}{I_g}$ $f_T := \dfrac{119.5\cdot(14)\cdot 12}{21952}$ $\boxed{f_T = 0.91}$ ksi

f_T =0.91 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

Knowing $A_s = 1.65 \dfrac{in^2}{ft}$ (required for strength)

Try: #8 at 5 1/2" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in (See Figure E18.4) $\boxed{h = 28}$ in (See Figure E18.4) $\boxed{\beta_s = 1.14}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.2.2, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 35.94}$ ksi $\leq 0.6\ f_y$ O.K. $s \leq \dfrac{700\cdot(0.75)}{1.14\cdot(35.94)} - 2\cdot(2.50) = 12.8 - 5.0 = 7.8$ in

$s \leq 7.8$ in

Therefore, spacing prov'd. = 5 1/2 in < 7.8 in <u>O.K.</u>

To insure that the reinforcement has the moment capacity to handle the Wisconsin Standard Permit Vehicle (Wis-SPV), the spacing was reduced to 5 inches. (See E18-1.8)

<u>Use: #8 at 5" c-c spacing at C/L Pier</u> (Max. negative reinforcement), $A_s = 1.88 \dfrac{in^2}{ft}$

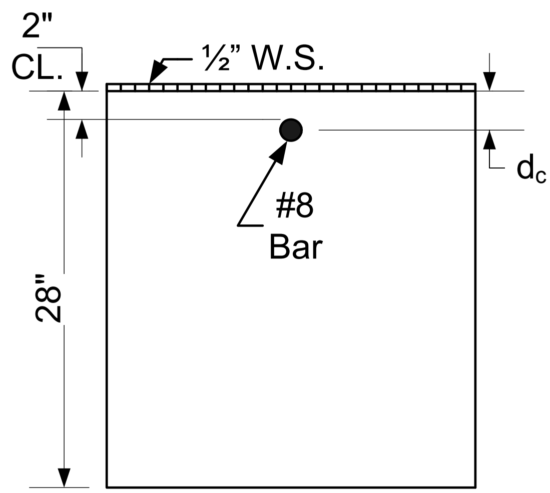

Figure E18.4

Cross Section - (at C/L of Pier)

E18-1.7.2.4 Minimum Reinforcement Check

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD [5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\dfrac{I_g}{c}$

Where:

$f_r = 0.24\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.48}$ ksi

$I_g := \dfrac{1}{12}\cdot b\cdot D_{haunch}^{\ 3}$ $\boxed{I_g = 21952}$ in^4 $c := \dfrac{D_{haunch}}{2}$ $\boxed{c = 14}$ in

$M_{cr} = \dfrac{1.1f_r\cdot(I_g)}{c} = \dfrac{1.1\cdot 0.48\cdot(21952)}{14(12)}$ $\boxed{M_{cr} = 68.99}$ kip-ft

$1.33\cdot M_u = 237.1$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.2.1 and (M_u = 178.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

By examining E18-1.7.2.1, the reinforcement yields, therefore:

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$ $\boxed{M_r = 204.1}$ kip-ft

Therefore, $M_{cr} = 68.99$ kip-ft $<$ $M_r = 204.1$ kip-ft O.K.

E18-1.7.3 Positive Moment Reinforcement for Span 2

Examine the 0.5 point of span 2

E18-1.7.3.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\ A_s\ f_s\ (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.5 pt.) of span 2:

$M_{DC} = 19.6$ kip-ft $M_{DW} = 1.6$ kip-ft $M_{LL+IM} = 8.2 + 37.4 = 45.6$ kip-ft

$M_u := 1.25 \cdot (19.6) + 1.50 \cdot (1.6) + 1.75 \cdot (45.6)$ $\boxed{M_u = 106.7}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 534}$ psi $\boxed{\rho = 0.0097}$ $\boxed{A_s = 1.73}$ $\frac{in^2}{ft}$

Try: #9 at 6" c-c spacing ($A_s = 2.00\ in^2/ft$) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.94}$ in

Then, $\boxed{c = 3.46}$ in and $\frac{c}{d_s} = 0.23 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 120.9}$ kip-ft

Therefore, $M_u = 106.7$ kip-ft $<$ $M_r = 120.9$ kip-ft O.K.

E18-1.7.3.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.5\cdot(f_{range}) \le 24 - 0.33\cdot f_{min}$ (for f_y = 60 ksi)

From Table E18.4, the moments at (0.5 pt.) of span 2 are:

M_{DC} = 19.6 kip-ft M_{DW} = 1.6 kip-ft

+Fatigue Truck = 16.7 kip-ft -Fatigue Truck = -3.4 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

Allowable tensile stress for fatigue (cracking stress): $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) +1.5(Fatigue Truck)

$\boxed{M_{fatigueMax} = 46.25}$ kip-ft (tension) $\boxed{M_{fatigueMin} = 16.1}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$, where $\boxed{y = 8.5}$ in $\boxed{I_g = 4913}$ in^4
:

$$f_{fatigue} = \frac{M_{fatigue}\cdot(y)}{I_g}$$

$\boxed{f_{fatigueMax} = 0.96}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$\boxed{f_{fatigueMin} = 0.33}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (+ Fatigue Truck) - (-Fatigue Truck) $\boxed{M_{range} = 20.1}$ kip-ft

The values for A_s, d_s, n and j (from transformed section) used to calculate f_{range} and f_{min} are:

$A_s = 1.73\ \frac{in^2}{ft}$ (required for strength), $d_s = 14.9$ in , $n := 8$, $j := 0.892$

The values for f_{range}, $f_{range1.5}$, and f_{min} are:

$\boxed{f_{range} = 10.43}$ ksi $\boxed{f_{range1.5} = 15.64}$ ksi $\boxed{f_{min} = 8.4}$ ksi

The factored resistance is: $\boxed{R_r = 21.23}$ ksi

Therefore, $1.5\cdot(f_{range})$ = 15.64 ksi < R_r = 21.23 ksi <u>O.K.</u>

E18-1.7.3.3 Check Crack Control

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.3.1, at (0.5 pt.) of span 2 provides:

M_{DC} = 19.6 kip-ft M_{DW} = 1.6 kip-ft M_{LL+IM} = 8.2 + 37.4 = 45.6 kip-ft (LL#1)

$M_s := 1.0 \cdot (19.6) + 1.0 \cdot (1.6) + 1.0 \cdot (45.6)$ $\boxed{M_s = 66.8}$ kip-ft

$f_T = \dfrac{M_s \cdot c}{I_g}$ $f_T := \dfrac{66.8 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 1.39}$ ksi

f_T =1.39 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

Knowing $A_s = 1.73 \ \dfrac{in^2}{ft}$ (required for strength)

Try: #9 at 6" c-c spacing (A_s = 2.00 in^2/ft) from Table 18.4-4 in 18.4.13

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

$\boxed{d_c = 2.064}$ in (See Figure E18.5) $\boxed{h = 17}$ in (See Figure E18.5) $\boxed{\beta_s = 1.2}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq$ 0.6 f_y

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.3.2, j = 0.892

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 30.08}$ ksi $\leq$ 0.6 f_y O.K. $s \leq \dfrac{700 \cdot (1.00)}{1.2 \cdot (30.08)} - 2 \cdot (2.064) = 19.4 - 4.1 = 15.3$ in

$s \leq 15.3$ in

Therefore, spacing prov'd. = 6 in < 15.3 in O.K.

Use: #9 at 6" c-c spacing in span 2 (Max. positive reinforcement).

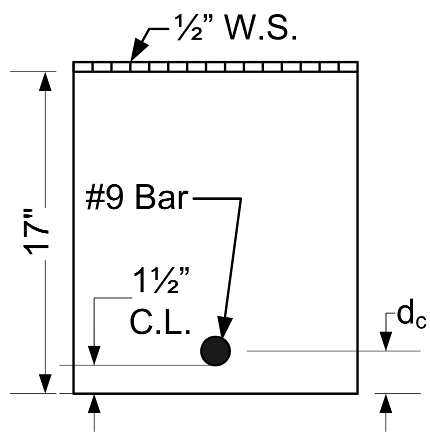

Figure E18.5

Cross Section - (0.5 pt.) Span 2

E18-1.7.3.4 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

E18-1.7.4 Negative Moment Reinforcement at Haunch/Slab Intercepts

Check the longitudinal reinforcement required at the C/L of the pier, to see if its adequate at the haunch/slab intercepts.

The haunch/slab intercepts are at (0.789 pt.) of span 1 and (0.157/0.843 pt.) of span 2. Moments at these locations are shown in Table E18.4.

Check #8 at 5" c-c spacing (as req'd. at Pier); $A_s := 1.88 \frac{in^2}{ft}$

Check for Strength:

Following the procedure in E18-1.7.1.1, using Strength I Limit State O.K.

Check for Fatigue:

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State O.K.

Check Crack Control:

Following the procedure in E18-1.7.1.3, using Service I Limit State O.K.

Minimum Reinforcement Check:

Following the procedure in E18-1.7.1.4 O.K.

E18-1.7.5 Bar Steel Cutoffs

Select longitudinal reinforcement cutoff locations for an Interior Strip.

E18-1.7.5.1 Span 1 Positive Moment Reinforcement (Cutoffs)

Theoretical bar steel cutoff points for positive moment are determined when one-half the steel required at the (0.4 pt.) has the moment capacity, or factored resistance , M_r , equal to the total factored moment , M_u , at these points. However, the remaining bars are to be extended beyond the theoretical cutoff point and must meet the fatigue and crack control requirements at these cutoff locations. The factored moments, M_u , at the 0.1 points and haunch/slab intercepts have been plotted on Figure E18.6. The capacities, M_r , of #9 at 7" and #9 at 14" are also shown. The factored moments, M_u , and capacities, M_r , are based on Strength I Limit State criteria. The positive live load moments, M_{LL+IM} , used to calculate M_u are taken as the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM). Use maximum or minimum load factors for M_{DC} and M_{DW} (See Table E18.1) to calculate the critical force effect. When value of M_{DW} is (-), assume FWS is not present and ignore it.

Calculate the capacity of #9 at 7" c-c spacing $A_s := 1.71\ \frac{in^2}{ft}$ $d_s := 14.9$ in

$b = 12$ inches (for a one foot design width)

As shown in E18-1.7.1.1, reinforcement will yield, therefore: $\boxed{a = 2.51}$ in

$$M_r := 0.9 \cdot (1.71) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{2.51}{2}}{12} \right) \qquad \boxed{M_r = 105} \text{ kip-ft}$$

Calculate the capacity of #9 at 14" c-c spacing $A_s := 0.86\ \frac{in^2}{ft}$ $d_s := 14.9$ in

For same section depth and less steel, reinforcement will yield, therefore: $\boxed{a = 1.26}$ in

$$M_r := 0.9 \cdot (0.86) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{1.26}{2}}{12} \right) \qquad \boxed{M_r = 55.2} \text{ kip-ft}$$

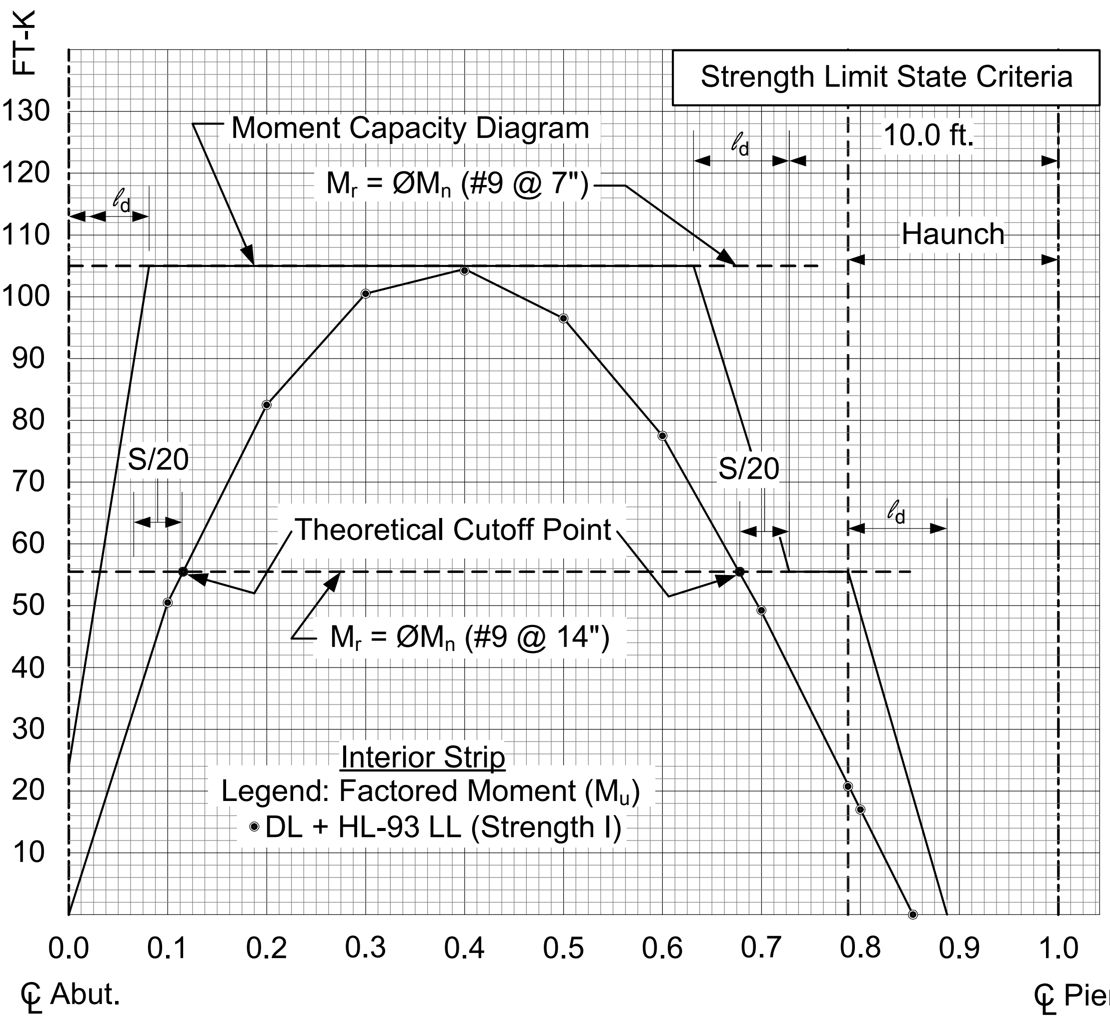

Figure E18.6

Span (1) - Positive Moment Cutoff Diagram

The moment diagram equals the capacity of #9 at 14" at 4.2 (ft) from the C/L of the abutment. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.11.1.2.1]**

$d_{eff} := 14.9$ in $\qquad\qquad$ ℓ_d (#9) (See Table 9.9-2, Chapter 9).

$15 \cdot (d_b) = 15 \cdot (1.128) = 16.9$ in

$\frac{S}{20} = \frac{38}{20} = 1.9$ ft $\qquad$ controls

Therefore, 1/2 of bars may be cut at 2.0 (ft) from the C/L of the abutment if fatigue and crack control criteria are satisfied.

Because the cutoff point is close to the abutment, don't cut 1/2 of bars, but run all #9 bars into the support. **LRFD [5.11.1.2.2]**

The moment diagram equals the capacity of #9 at 14" at 12.1 (ft) from the C/L of pier. Reinforcement shall be extended S/20 beyond this point.

Therefore, 1/2 of bars may be cut at 10.0 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.74 pt.)

E18-1.7.5.1.1 Fatigue Check (at Cutoff) - (0.74 Pt.)

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.5 \cdot (f_{range}) \le 24 - 0.33 \cdot f_{min}$ $\qquad$ (for f_y = 60 ksi)

Interpolating from Table E18.4, the moments at (0.74 pt.) of span 1 are:

M_{DC} = -10.0 kip-ft $\qquad$ M_{DW} = -0.89 kip-ft

+Fatigue Truck = 9.72 kip-ft $\qquad$ -Fatigue Truck = -10.34 kip-ft

In regions of compressive stress due to permanent loads, fatigue shall be considered only if this compressive stress is less than ($\gamma_{LLfatigue}$ = 1.5) times the maximum tensile stress from the fatigue load. **LRFD [5.5.3.1]**

For simplicity, assume fatigue criteria should be checked.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0 \cdot (-10.0) + 1.0(-0.89) + 1.5(9.72)$ $\qquad$ $\boxed{M_{fatigueMax} = 3.69}$ kip-ft (tens.)

$M_{fatigueMin} := 1.0 \cdot (-10.0) + 1.0(-0.89) + 1.5(-10.34)$ $\qquad$ $\boxed{M_{fatigueMin} = -26.4}$ kip-ft (compr.)

Looking at values of $M_{fatigue}$ shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle.

Following the procedure outlined in E18-1.7.5.2.1, fatigue criteria at bar cutoff is O.K.

E18-1.7.5.1.2 Crack Control Check (at Cutoff) - (0.74 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.74 pt.) of span 1 are:

M_{DC} = -10.0 kip-ft M_{DW} = -0.89 kip-ft M_{LL+IM} = 4.7 + 21.1 = 25.8 kip-ft (LL#1)

$M_s := 1.0 \cdot (-10.0) + 1.0 \cdot (25.8)$ $\boxed{M_s = 15.8}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment

$f_T = \frac{M_s \cdot c}{I_g}$ $f_T := \frac{15.8 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 0.33}$ ksi

f_T =0.33 ksi < 80% f_r = 0.38 ksi; therefore, crack control criteria check not req'd.

Therefore, crack control criteria at bar cutoff is O.K.

E18-1.7.5.1.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

Therefore cut 1/2 of bars at 10.0 (ft) from the C/L of pier. Remaining bars are extended (ℓ_d) beyond the haunch/slab intercept as shown on Standard 18.01.

E18-1.7.5.2 Span 2 Positive Moment Reinforcement (Cutoffs)

Theoretical bar steel cutoff points for positive moment are determined when one-half the steel required at the (0.5 pt.) has the moment capacity, or factored resistance , M_r , equal to the total factored moment , M_u , at these points. However, the remaining bars are to be extended beyond the theoretical cutoff point and must meet the fatigue and crack control requirements at these cutoff locations. The factored moments, M_u , at the 0.1 points and haunch/slab intercepts have been plotted on Figure E18.7. The capacities, M_r , of #9 at 6" and #9 at 12" are also shown. The factored moments, M_u , and capacities, M_r , are based on Strength I Limit State criteria. The positive live load moments, M_{LL+IM} , used to calculate M_u are taken as the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM). Use maximum or minimum load factors for M_{DC} and M_{DW} (See Table E18.1) to calculate the critical force effect. When value of M_{DW} is (-), assume FWS is not present and ignore it.

Calculate the capacity of #9 at 6" c-c spacing $A_s := 2.00 \, \frac{in^2}{ft}$ $d_s := 14.9$ in

$b = 12$ inches (for a one foot design width)

As shown in E18-1.7.3.1, reinforcement will yield, therefore: $\boxed{a = 2.94}$ in

$$M_r := 0.9 \cdot (2.00) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{2.94}{2}}{12} \right) \qquad \boxed{M_r = 120.9} \text{ kip-ft}$$

Calculate the capacity of #9 at 12" c-c spacing $A_s := 1.00 \, \frac{in^2}{ft}$ $d_s := 14.9$ in

For same section depth and less steel, reinforcement will yield, therefore: $\boxed{a = 1.47}$ in

$$M_r := 0.9 \cdot (1.00) \cdot 60.0 \cdot \left(\frac{14.9 - \frac{1.47}{2}}{12} \right) \qquad \boxed{M_r = 63.7} \text{ kip-ft}$$

The moment diagram equals the capacity of #9 at 12" at 14.4 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.11.1.2.1]**

$\frac{S}{20} = \frac{51}{20} = 2.55$ ft <u>controls</u> ℓ_d (#9) (See Table 9.9-2, Chapter 9).

Therefore, 1/2 of bars may be cut at 11.5 (ft) from the C/L of each pier if fatigue and crack control criteria are satisfied (Check at 0.23 pt.).

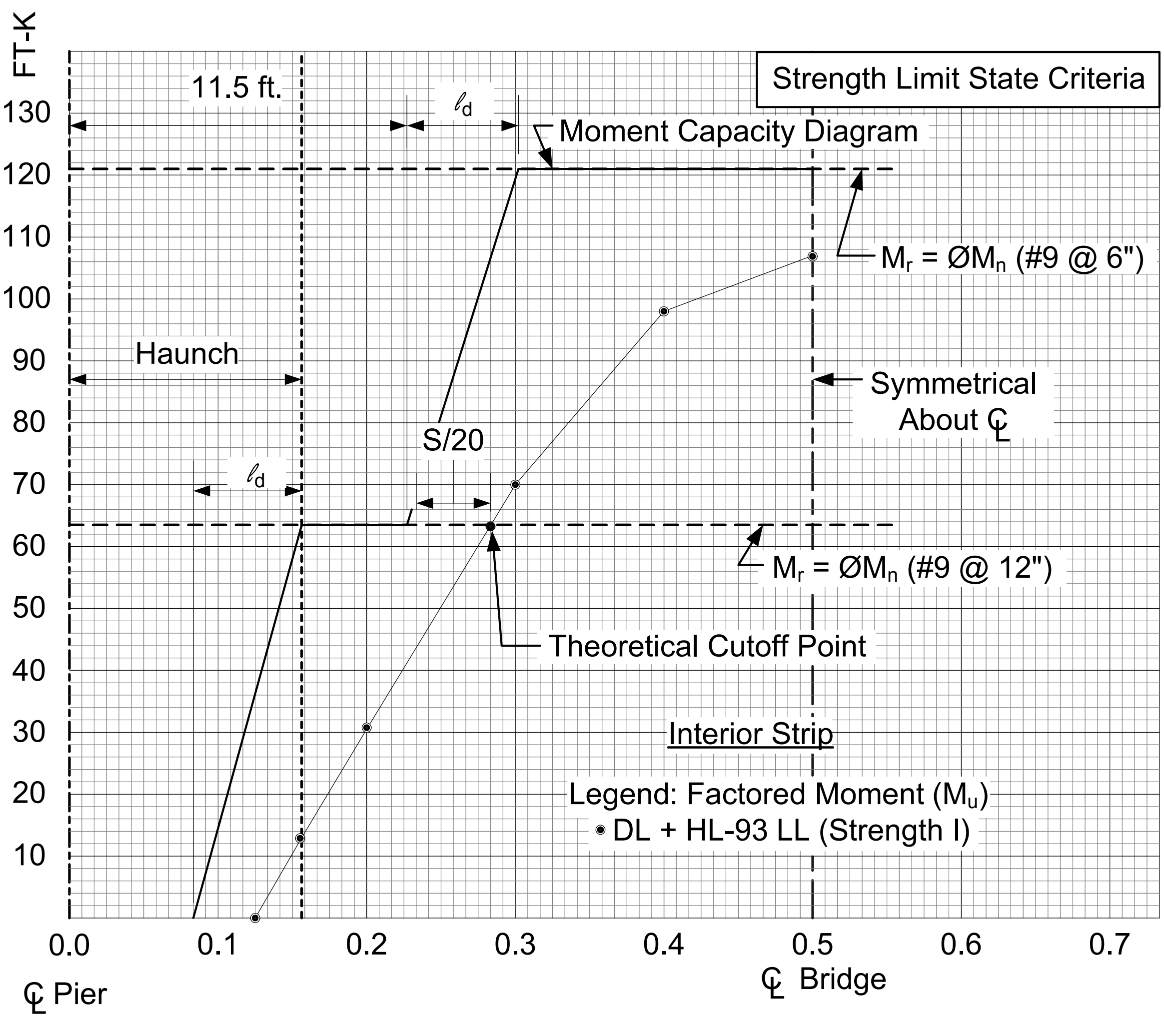

Figure E18.7

Span (2) - Positive Moment Cutoff Diagram

E18-1.7.5.2.1 Fatigue Check (at Cutoff) - (0.23 Pt.)

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLfatigue} := 1.5$ $\phi_{fatigue} := 1.0$

When reinforcement goes through tensile and compressive stress during the fatigue cycle,

$$Q = f_s + f'_s$$

Where:

f_s = tensile part of stress range in bar reinforcement due to dead load moments from applied loads in E18-1.2 and largest factored tensile moment caused by Fatigue Truck (LL#4)

f'_s = compressive part of stress range in bar reinforcement due to dead load moments from applied loads in E18-1.2 and largest factored compressive moment caused by Fatigue Truck (LL#4)

All live load moments in f_s and f'_s are multiplied by (η_i) and ($\gamma_{LLfatigue}$)

See Table E18.2 and E18.3 in E18-1.4 for description of live load and dynamic load allowance (IM).

$R_n = (\Delta F_{TH}) = 24 - 0.33 \cdot f_{min}$ (for f_y = 60 ksi) (See 18.3.5.2.1)

$R_r = \phi_{fatigue} \cdot R_n = 1.0 \cdot (24 - 0.33 \cdot f_{min})$

Therefore: $f_s + f'_s \leq 24 - 0.33 \cdot f_{min}$ (Limit States Equation)

Interpolating from Table E2, the moments at (0.23 pt.) of span 2 are:

M_{DC} = -3.5 kip-ft M_{DW} = -0.31 kip-ft

+Fatigue Truck = 10.02 kip-ft -Fatigue Truck = -7.3 kip-ft

In regions of compressive stress due to permanent loads, fatigue shall be considered only if this compressive stress is less than ($\gamma_{LLfatigue}$ = 1.5) times the maximum tensile stress from the fatigue load. **LRFD [5.5.3.1]**

The section properties for fatigue shall be based on cracked sections where the sum of stresses, due to permanent loads, and ($\gamma_{LLfatigue}$ = 1.5) times the fatigue load is tensile and exceeds $0.095\sqrt{f'_c}$.

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(Fatigue Truck)

$M_{fatigueMax} := 1.0(-3.5) + 1.5(10.02)$ $\boxed{M_{fatigueMax} = 11.53}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(-3.5) + 1.5(-7.3)$ $\boxed{M_{fatigueMin} = -14.45}$ kip-ft (compression)

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile range.

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.8, for definition of d_1, d_2, d', A_s and A'_s.

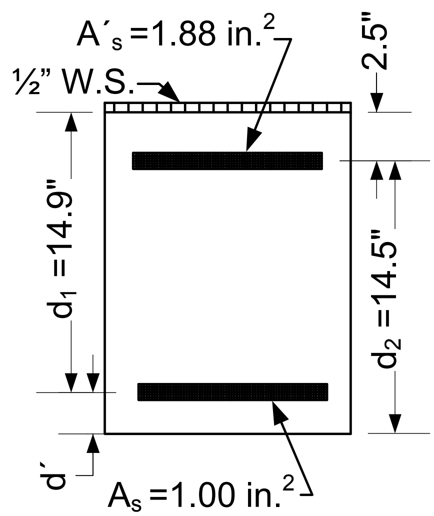

Figure E18.8

Cross Section - (0.23 pt.) Span 2

The moment arm used in equations below is: $(j_1)\,(d_1)$ for finding f_s
$(j_2)\,(d_2)$ for finding f'_s

Using: A_s = 1.00 in²/ft, d_1 = 14.9 in, n = 8, and transformed section analysis, gives a value of j_1 = 0.914

Using: A'_s = 1.88 in²/ft, d_2 = 14.5 in, n = 8, and transformed section analysis, gives a value of j_2 = 0.887; k = x/d_2 = 0.34, where x = distance from compression face to neutral axis

The tensile part of the stress range in the bottom bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1}$$ $\boxed{f_s = 10.16}$ ksi (tension)

The compressive part of the stress range in the bottom bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k}$$ $\boxed{f'_s = -2.15}$ ksi (compression)

It is assumed (#8's at 5") req'd. at pier, is present at this location as compression steel (A'_s).

Therefore, total stress range on bottom steel:

$f_s + f'_s = 10.16 - (-2.15) = 12.31$ ksi

$R_r := 24 - 0.33 \cdot f_{min}$ where $f_{min} = f'_s$, therefore: $\boxed{R_r = 24.71}$ ksi

Therefore, $f_s + f'_s = 12.31$ ksi < $R_r = 24.71$ ksi O.K..

E18-1.7.5.2.2 Crack Control Check (at Cutoff) - (0.23 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.23 pt.) of span 2 are:

M_{DC} = -3.51 kip-ft M_{DW} = -0.31 kip-ft M_{LL+IM} = 3.7 + (21.9) = 25.6 kip-ft (LL#1)

$M_s := 1.0 \cdot (-3.51) + 1.0 \cdot (25.6)$ $\boxed{M_s = 22.1}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment.

$$f_T = \frac{M_s \cdot c}{I_g} \qquad f_T := \frac{22.1 \cdot (8.5) \cdot 12}{4913} \qquad \boxed{f_T = 0.46} \text{ ksi}$$

f_T =0.46 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

For: #9 at 12" c-c spacing ($A_s = 1.00$ in^2/ft)

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

$\boxed{d_c = 2.064}$ in $\boxed{h = 17}$ in $\boxed{\beta_s = 1.2}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)

As shown in fatigue calculations in E18-1.7.5.2.1, j = 0.914

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 19.43}$ ksi $\leq 0.6\,f_y$ O.K. $\quad s \leq \dfrac{700\cdot(1.00)}{1.2\cdot(19.43)} - 2\cdot(2.064) = 30.07 - 4.1 = 26.0$ in

$s \leq 26.0$ in

Therefore, spacing prov'd. = 12 in < 26.0 in O.K.

E18-1.7.5.2.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 11.5 (ft) from the C/L of each pier. Remaining bars are extended (ℓ_d) beyond the haunch/slab intercept as shown on Standard 18.01.

E18-1.7.5.3 Span 1 Negative Moment Reinforcement (Cutoffs)

Theoretical bar steel cutoff points for negative moment are determined when one-half the steel required at the (C/L Pier) has the moment capacity, or factored resistance , M_r , equal to the total factored moment , M_u , at these points. However, the remaining bars are to be extended beyond the theoretical cutoff point and must meet the fatigue and crack control requirements at these cutoff locations. The factored moments, M_u , at the 0.1 points and haunch/slab intercepts have been plotted on Figure E18.9. The capacities, M_r , of #8 at 5" and #8 at 10" are also shown. The factored moments, M_u , and capacities, M_r , are based on Strength I Limit State criteria. The negative live load moments, M_{LL+IM} , used to calculate M_u are taken as the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM). Use maximum or minimum load factors for M_{DC} and M_{DW} (See Table E18.1) to calculate the critical force effect. When value of M_{DW} is (+), assume FWS is not present and ignore it.

Calculate the capacity of #8 at 5" c-c spacing $\quad A_s := 1.88 \ \dfrac{in^2}{ft}$

$b = 12$ inches (for a one foot design width)

As shown in E18-1.7.2.1, reinforcement will yield, therefore: $\boxed{a = 2.76}$ in

$M_r = 204.1$ kip-ft (at C/L pier), $\quad d_s := 25.5$ in

$M_r = 111.0$ kip-ft (in span), $\quad d_s := 14.5$ in

Calculate the capacity of #8 at 10" c-c spacing $\quad A_s := 0.94 \ \dfrac{in^2}{ft}$

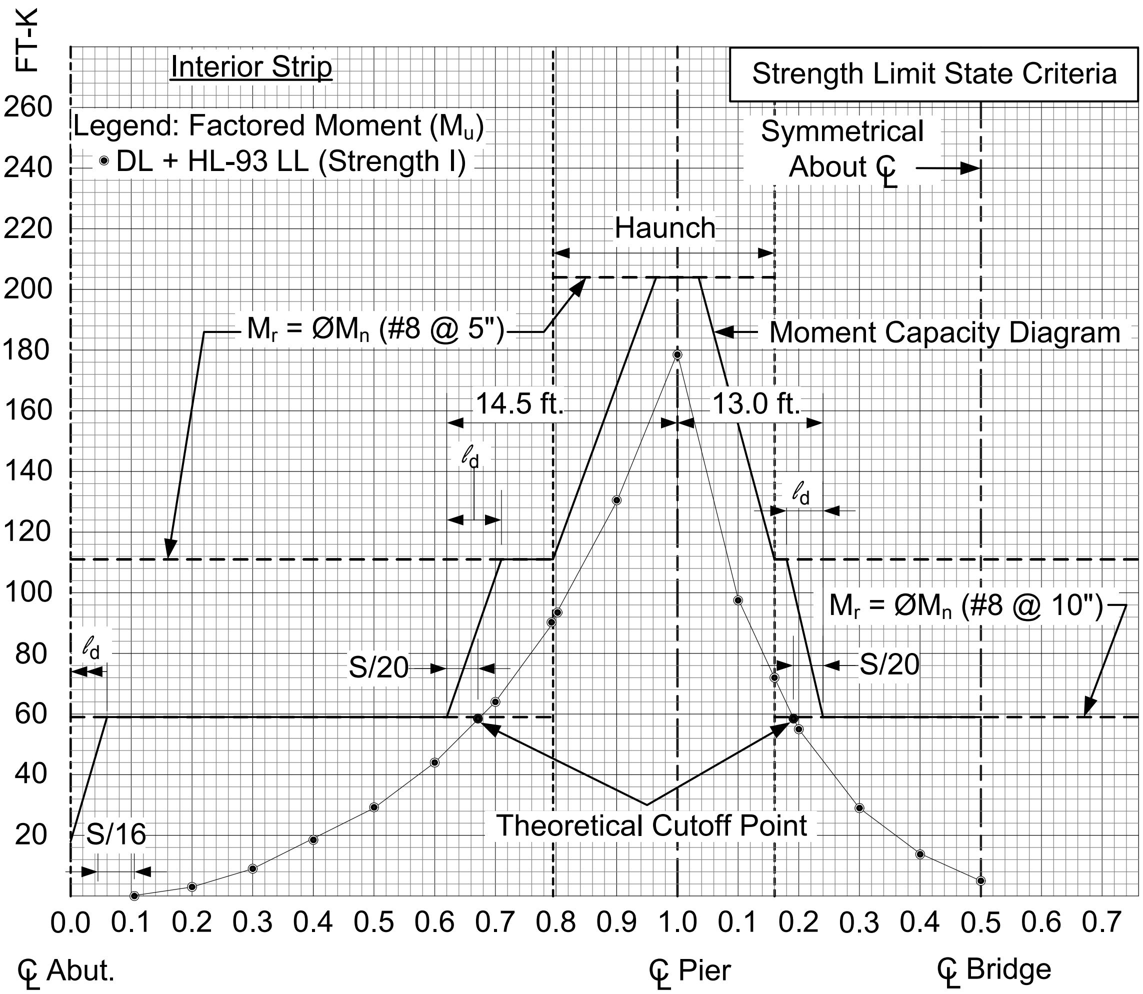

Figure E18.9

Negative Moment Cutoff Diagram

For same section depth and less steel, reinforcement will yield, therefore:

$\boxed{a = 1.38}$ in

$M_r = 104.9$ kip-ft (at C/L pier), $d_s := 25.5$ in

$M_r = 58.4$ kip-ft (in span), $d_s := 14.5$ in

The moment diagram equals the capacity of #8 at 10" at 12.5 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.11.1.2.1]**

$\frac{S}{20} = \frac{38}{20} = 1.9$ ft <u>controls</u> ℓ_d (#8) (See Table 9.9-2, Chapter 9)

Therefore, 1/2 of bars may be cut at 14.5 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.62 pt.)

E18-1.7.5.3.1 Fatigue Check (at Cutoff) - (0.62 Pt.)

Following the procedure in E18-1.7.5.2.1, using Fatigue I Limit State:

$f_s + f'_s \le 24 - 0.33 \cdot f_{min}$ (for f_y = 60 ksi)

Interpolating from Table E18.4, the moments at (0.62 pt.) of span 1 are:

M_{DC} = 4.44 kip-ft M_{DW} = 0.4 kip-ft

+Fatigue Truck = 13.7 kip-ft   -Fatigue Truck = -8.68 kip-ft

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0(4.44) + 1.5(-8.68)$ $\boxed{M_{fatigueMax} = -8.58}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(4.44) + 1.5(13.7)$ $\boxed{M_{fatigueMin} = 24.99}$ kip-ft (compression)

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile range.

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.10, for definition of d_1, d_2, d', A_s and A'_s.

The moment arm used in equations below is: $(j_1)(d_1)$ for finding f_s
$(j_2)(d_2)$ for finding f'_s

Using: A_s = 0.94 in^2/ft, d_1 = 14.5 in, n = 8, and transformed section analysis, gives a value of j_1 = 0.915

Using: A'_s =1.71 in^2/ft, d_2 = 14.9 in, n = 8, and transformed section analysis, gives a value of j_2 = 0.893; k = x/d_2 = 0.33, where x = distance from compression face to neutral axis

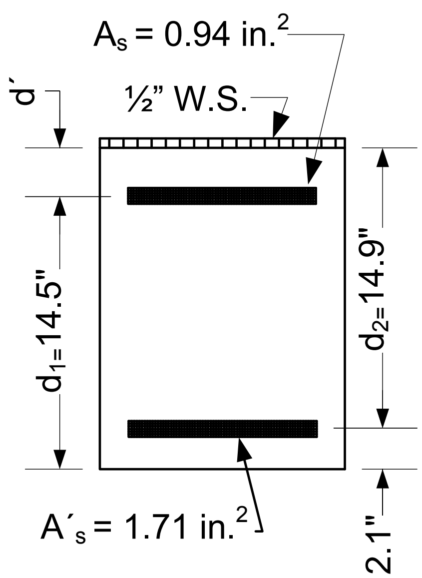

Figure E18.10

Cross Section - (0.62 pt.) Span 1

The tensile part of the stress range in the top bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1} \qquad \boxed{f_s = 8.26} \quad \text{ksi} \quad \text{(tension)}$$

The compressive part of the stress range in the top bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k} \qquad \boxed{f'_s = -3.19} \quad \text{ksi} \quad \text{(compression)}$$

It is assumed (#9's at 7") is present at this location as compression steel (A'_s).

Therefore, total stress range on top steel:

$$f_s + f'_s = 8.26 - (-3.19) = 11.45 \quad \text{ksi}$$

$$R_r := 24 - 0.33 \cdot f_{min} \quad \text{where } f_{min} = f'_s \text{, therefore:} \quad \boxed{R_r = 25.05} \quad \text{ksi}$$

Therefore, $f_s + f'_s$ = 11.45 ksi < R_r = 25.05 ksi   O.K..

E18-1.7.5.3.2 Crack Control Check (at Cutoff) - (0.62 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.62 pt.) of span 1 are:

$M_{DC} = 4.4$ kip-ft $M_{DW} = 0.4$ kip-ft $M_{LL+IM} = -5.88 + (-23.88) = -29.8$ kip-ft (LL#2)

$M_s := 1.0 \cdot (4.4) + 1.0 \cdot (-29.8)$ $\boxed{M_s = -25.4}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment.

$f_T = \frac{M_s \cdot c}{I_g}$ $f_T := \frac{25.4 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 0.53}$ ksi

f_T =0.53 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

For: #8 at 10" c-c spacing ($A_s = 0.94\ in^2/ft$)

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in $\boxed{h = 17}$ in $\boxed{\beta_s = 1.25}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.5.3.1, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 24.44}$ ksi < 0.6 f_y O.K. $s \leq \frac{700 \cdot (0.75)}{1.25 \cdot (24.44)} - 2 \cdot (2.50) = 17.2 - 5.0 = 12.2$ in

$s \leq 12.2$ in

Therefore, spacing prov'd. = 10 in < 12.2 in O.K.

E18-1.7.5.3.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 14.5 (ft) from the C/L of the pier. Remaining bars are extended beyond the point of inflection, a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater. **LRFD [5.11.1.2.3]**

$d_{eff} := 14.5$ in $\qquad\qquad$ ℓ_d (#8) (See Table 9.9-2, Chapter 9)

$$12 \cdot (d_b) = 12 \cdot (1.00) = 12.0 \text{ in}$$

$$\frac{S}{16} = \frac{38}{16} = 2.38 \text{ ft}$$ controls

Looking at the factored moment diagram (M_u) on Figure E18.9, the point of inflection is found at the (0.11 pt.). Therefore, the remaining bars could be terminated at 36.5 (ft) from the C/L of pier and these bars lapped with smaller size bars spaced at 10 inches.

Because this bar termination point is close to the abutment, run remaining bars (#8 at 10" c-c spacing) to the end of the slab.

E18-1.7.5.4 Span 2 Negative Moment Reinforcement (Cutoffs)

Capacities of #8 at 5" and #8 at 10" c-c spacing are stated in E18-1.7.5.3

The moment diagram equals the capacity of #8 at 10 " at 10.0 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.11.1.2.1]**

$$\frac{S}{20} = \frac{51}{20} = 2.55 \text{ ft}$$ controls $\qquad$ ℓ_d (#8) (See Table 9.9-2, Chapter 9)

Therefore, 1/2 of bars may be cut at 13.0 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.25 pt.)

E18-1.7.5.4.1 Fatigue Check (at Cutoff) - (0.25 Pt.)

Following the procedure in E18-1.7.5.2.1, using Fatigue I Limit State:

$$f_s + f'_s \le 24 - 0.33 \cdot f_{min} \qquad \text{(for } f_y = 60 \text{ ksi)}$$

Interpolating from Table E18.4, the moments at (0.25 pt.) of span 2 are:

M_{DC} = -0.45 kip-ft $\qquad$ M_{DW} = -0.05 kip-ft

+Fatigue Truck = 10.9 kip-ft $\qquad$ -Fatigue Truck = -7.0 kip-ft

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0(-0.45) + 1.0(-0.05) + 1.5(-7.0)$ $\boxed{M_{fatigueMax} = -11.0}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(-0.45) + 1.0(-0.05) + 1.5(10.9)$ $\boxed{M_{fatigueMin} = 15.85}$ kip-ft (compr.)

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.11, for definition of d_1, d_2, d', A_s and A'_s.

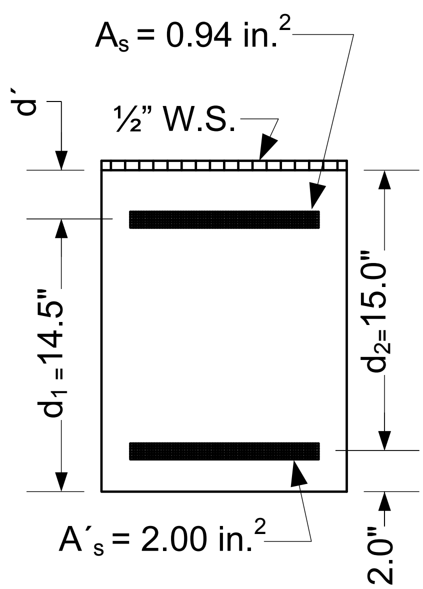

Figure E18.11

Cross Section - (0.25 pt.) Span 2

The moment arm used in equations below is: $(j_1)(d_1)$ for finding f_s
$(j_2)(d_2)$ for finding f'_s

Using: A_s = 0.94 in^2/ft, d_1 = 14.5 in, n = 8, and transformed section analysis, gives a value of $j_1 = 0.915$

Using: A'_s =2.00 in^2/ft, d_2 = 15.0 in, n = 8, and transformed section analysis, gives a value of $j_2 = 0.886$; $k = x/d_2 = 0.34$, where x = distance from compression face to neutral axis

The tensile part of the stress range in the top bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1}$$ $\boxed{f_s = 10.58}$ ksi (tension)

The compressive part of the stress range in the top bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k}$$ $\boxed{f'_s = -1.88}$ ksi (compression)

It is assumed (#9's at 6") is present at this location as compression steel (A'_s).

Therefore, total stress range on top steel:

$$f_s + f'_s = 10.58 - (-1.88) = 12.46 \quad \text{ksi}$$

$R_r := 24 - 0.33 \cdot f_{min}$ where $f_{min} = f'_s$, therefore: $\boxed{R_r = 24.62}$ ksi

Therefore, $f_s + f'_s = 12.46$ ksi < $R_r = 24.62$ ksi <u>O.K..</u>

E18-1.7.5.4.2 Crack Control Check (at Cutoff) - (0.25 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.25 pt.) of span 2 are:

M_{DC} = -0.45 kip-ft M_{DW} = -0.05 kip-ft M_{LL+IM} = -4.35 + (-18.25) = -22.6 kip-ft (LL#2)

$M_s := 1.0 \cdot (0.45) + 1.0(0.05) + 1.0 \cdot (22.6)$ $\boxed{M_s = 23.1}$ kip-ft

$$f_T = \frac{M_s \cdot c}{I_g} \qquad f_T := \frac{23.1 \cdot (8.5) \cdot 12}{4913}$$ $\boxed{f_T = 0.48}$ ksi

f_T =0.48 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

For: #8 at 10" c-c spacing ($A_s = 0.94\ in^2/ft$)

The values for γ_e, d_c, h, and β_s, used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in $\boxed{h = 17}$ in $\boxed{\beta_s = 1.25}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.5.3.1, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 22.23}$ ksi < 0.6 f_y O.K.

$$s \leq \frac{700 \cdot (0.75)}{1.25 \cdot (22.23)} - 2 \cdot (2.50) = 19.0 - 5.0 = 14.0 \quad \text{in}$$

$s \leq 14.0$ in

Therefore, spacing prov'd. = 10 in < 14.0 in O.K.

E18-1.7.5.4.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 13.0 (ft) from the C/L of the pier. Remaining bars are extended beyond the point of inflection, a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater. **LRFD [5.11.1.2.3]**

$\frac{S}{16} = \frac{51}{16} = 3.19$ ft controls ℓ_d (#8) (See Table 9.9-2, Chapter 9)

Looking at the factored moment diagram (M_u) on Figure E18.9, no point of inflection is found in span 2.

Therefore, run the remaining bars (#8 at 10" c-c spacing) to the C/L of span 2 and lap them.

E18-1.8 Evaluation of Longitudinal Reinforcement for Permit Vehicle

Check the adequacy of the longitudinal reinforcement to see if it has the moment capacity to handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1.

The Wisconsin Standard Permit Vehicle load that can be carried by the bridge is 225 kips, when the future wearing surface is present. Details for the calculation of this load are shown in Chapter 45, "Reinforced Concrete Slab Rating" example.

Wisconsin Standard Permit Vehicle (Wis-SPV) load capacity = 225 kips > 190 kips O.K.

E18-1.9 Longitudinal Reinforcement in Bottom of Haunch

At least (1/4) of maximum positive moment reinforcement in continuous-spans shall extend into the support **LRFD [5.11.1.2.2]**.

Max. positive (A_s) = 2.00 $\frac{in^2}{ft}$ (#9 at 6" c-c spacing, in span 2)

Reinf. req'd. = $0.25 \cdot (2.00) = 0.5 \ \frac{in^2}{ft}$

Therefore, use #7 at 13 in. (0.55 in^2/ft) > reinf. req'd and min. reinf. on Standard 18.01 O.K.

See Figure E18.12 for a summary of longitudinal reinforcement selected and layout of transverse distribution steel selected in E18-1.12.

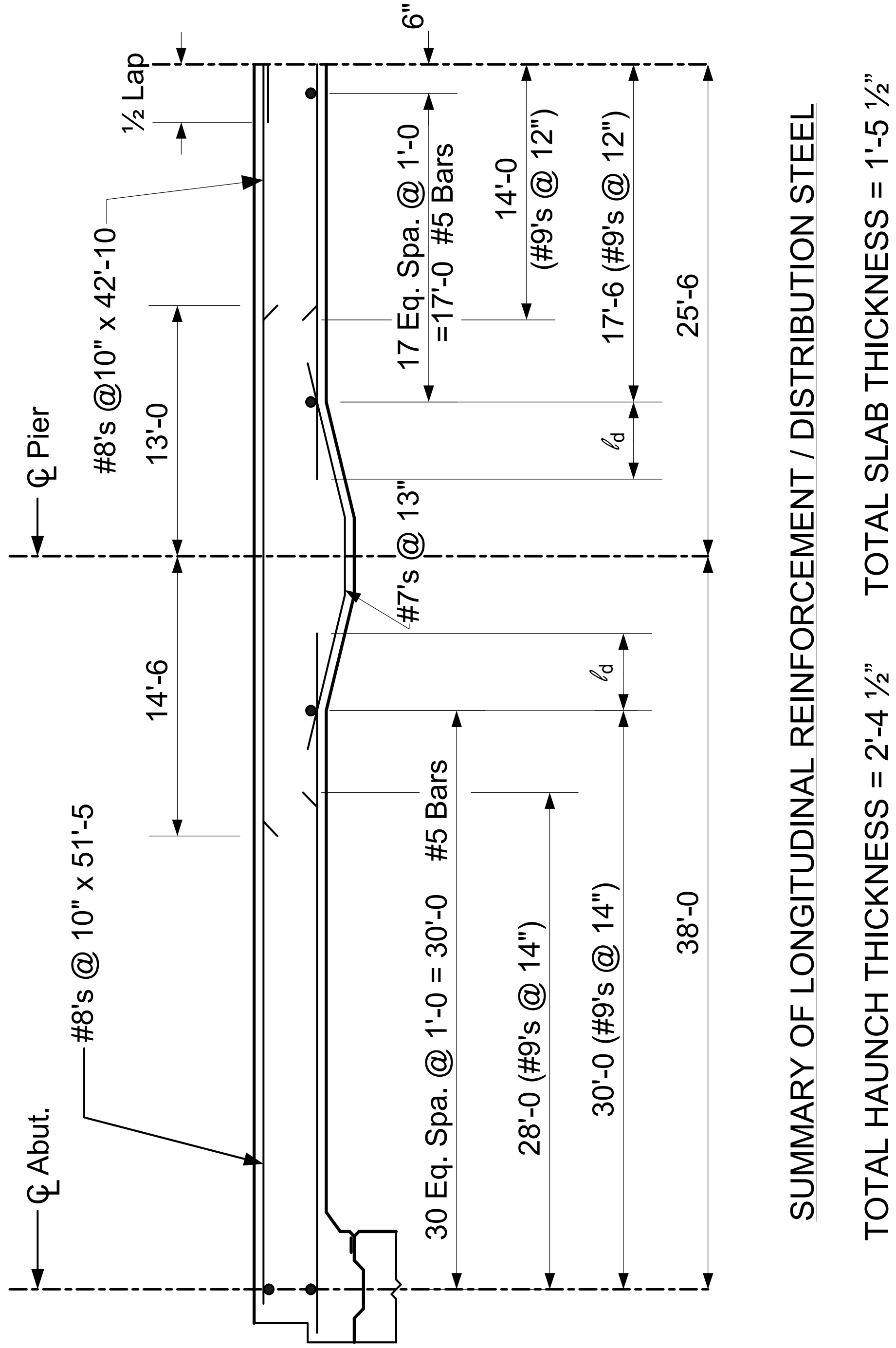

Figure E18.12

Summary of Longitudinal Reinforcement / Distribution Steel

E18-1.10 Live Load Distribution (Exterior Strip)

The exterior strip width (E), is assumed to carry one wheel line and a tributary portion of design lane load **LRFD [4.6.2.1.4]**.

(E) equals the distance between the edge of the slab and the inside face of the barrier, plus 12 inches, plus 1/4 of the full strip width specified in **LRFD [4.6.2.3]**.

The exterior strip width (E) shall not exceed either 1/2 the full strip width or 72 inches.

The distance from the edge of slab to the inside face of barrier = 15 inches

E18-1.10.1 Strength and Service Limit State

Use the smaller equivalent widths, which are from multi-lane loading, for full strip width when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

From previous calculations in E18-1.6:

Full strip width = 141 in. (Span 1,3) - multi-lane loading

Full strip width = 151 in. (Span 2) - multi-lane loading

The multiple presence factor (m) has been included in the equations for full strip width and therefore aren't used to adjust the distribution factor. **LRFD [3.6.1.1.2]**

Span 1, 3: $E = 15 + 12 + \frac{141}{4} = 62.2$ in.; but not to exceed ($\frac{141}{2}$ in. or 72 in.)

Therefore, E = 62.2 in. (Spans 1, 3)

Span 2: $E = 15 + 12 + \frac{151}{4} = 64.7$ in.; but not to exceed ($\frac{151}{2}$ in. or 72 in.)

Therefore, E = 64.7 in. (Span 2)

The distribution factor (DF) is computed for a design slab width equal to one foot.

Compute the distribution factor associated with one truck wheel line, to be applied to axle loads

$$DF = \frac{1wheel\_line}{\left(\frac{2wheel\_lines}{lane}\right) \cdot E}$$ (where E is in feet)

For Spans 1 & 3: (E = 62.2" = 5.183')

$$DF := \frac{1}{2 \cdot (5.183)}$$

$$\boxed{DF = 0.096} \; \frac{lanes}{ft - slab}$$

For Span 2: (E = 64.7" = 5.392')

$$DF := \frac{1}{2 \cdot (5.392)} \qquad \boxed{DF = 0.093} \ \frac{\text{lanes}}{\text{ft} - \text{slab}}$$

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to full lane load: **LRFD [3.6.1.2.4]**

$$DF = \left(\frac{\dfrac{SWL}{10ft\_lane\_load\_width}}{E} \right) \quad \text{(where E is in feet)}$$

SWL = slab width loaded = (E) - (distance from the edge of slab to inside face of barrier) (ft)

= 62.2 - 15 = 47.2 in. = 3.93 ft. (Span 1 & 3)

= 64.7 - 15 = 49.7 in. = 4.14 ft. (Span 2)

For Spans 1 & 3: (E = 5.183' ; SWL = 3.93')

$$DF := \frac{3.93 \div 10}{5.183} \qquad \boxed{DF = 0.076} \ \frac{\text{lanes}}{\text{ft} - \text{slab}}$$

For Span 2: (E = 5.392' ; SWL = 4.14')

$$DF := \frac{4.14 \div 10}{5.392} \qquad \boxed{DF = 0.077} \ \frac{\text{lanes}}{\text{ft} - \text{slab}}$$

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore, use : DF = 0.096 lanes/ft.-slab, for Design Truck and Design Tandem Loads

DF = 0.077 lanes/ft.-slab, for Design Lane Load

The concrete parapet is not to be considered to provide strength to the exterior strip (edge beam) **LRFD [9.5.1]**.

TABLE E18.5 Unfactored Moments (kip - ft) (on a one foot design width) **Exterior Strip**

Point	M_{DC} [1]	M_{DW} [2]	DF=0.077 (IM not used) +Design Lane	DF=0.077 (IM not used) -Design Lane	DF=0.096 (incl. IM =33%) +Design Tandem	DF=0.096 (incl. IM =33%) -Design Tandem
0.1	11.9	0.8	2.9	-0.9	19.4	-3.6
0.2	19.6	1.3	5.0	-1.7	32.7	-7.2
0.3	23.0	1.6	6.4	-2.6	40.0	-10.8
0.4	22.2	1.5	7.1	-3.4	42.3	-14.4
0.5	17.3	1.2	7.1	-4.3	40.8	-18.0
0.6	8.1	0.6	6.5	-5.2	36.0	-21.7
0.7	-5.3	-0.4	5.1	-6.0	27.9	-25.2
0.789	-21.1	-1.5	3.3	-6.9	19.0	-28.3
0.8	-22.9	-1.6	3.2	-7.1	17.8	-28.8
0.9	-45.0	-3.1	2.2	-9.8	9.5	-32.4
1.0	-72.6	-4.9	2.0	-14.0	10.4	-36.0
1.1	-36.7	-2.5	1.7	-8.0	8.6	-24.6
1.157	-20.8	-1.4	2.1	-5.6	15.6	-22.3
1.2	-10.1	-0.7	2.6	-4.4	21.3	-20.8
1.3	8.8	0.6	4.9	-3.4	32.6	-16.8
1.4	20.2	1.4	6.8	-3.4	39.9	-12.9
1.5	24.0	1.6	7.4	-3.4	42.2	-9.0

Point	DF=0.096 (incl. IM =33%) +Design Truck	DF=0.096 (incl. IM =33%) -Design Truck	DF=0.077 [3] (IM not used) (90%) of -Design Lane	DF=0.096 [3] (incl. IM =33%) (90%) of -Double Design Trucks
0.1	20.4	-4.4	------	------
0.2	33.1	-8.7	------	------
0.3	38.8	-13.1	------	------
0.4	39.9	-17.4	------	------
0.5	38.2	-21.8	------	------
0.6	34.6	-26.0	------	------
0.7	26.3	-30.5	-5.4	-27.4
0.789	15.8	-34.4	-6.2	-30.9
0.8	14.7	-34.9	-6.4	-31.4
0.9	10.2	-39.1	-8.8	-35.4
1.0	11.4	-45.0	-12.6	-39.5
1.1	9.0	-26.8	-7.2	-25.5
1.157	13.6	-24.5	-5.0	-22.8
1.2	17.3	-22.7	-4.0	-20.9
1.3	31.2	-18.5	------	------
1.4	39.9	-14.1	------	------
1.5	42.0	-9.9	------	------

Superscripts for Table E18.5 are defined on the following page.

In Table E18.5:

1 M_{DC} is moment due to slab dead load (DC_{slab}), parapet dead load (DC_{para}) after its weight is distributed across exterior strip width (E) and 1/2 inch wearing surface ($DC_{1/2"WS}$).

Using average of exterior strip widths: $\frac{62.2 + 64.7}{2}$ = 63.5 in = 5.3 ft

DC_{para} = (Parapet wgt.) / 5.3 ft = (387 plf) / 5.3 ft = 73 plf (on a 1'-0 slab width)

2 M_{DW} is moment due to future wearing surface (DW_{FWS})

3 The points of contraflexure are located at the (0.66 pt.) of span 1 and the (0.25 pt.) of span 2, when a uniform load is placed across the entire structure. Negative moments in these columns are shown between the points of contraflexure per **LRFD [3.6.1.3.1]**.

E18-1.11 Longitudinal Slab Reinforcement (Exterior Strip)

Select longitudinal reinforcement for an Exterior Strip (edge beam) **LRFD [5.14.4.1]**.

The reinforcement in the Exterior Strip is always equal to or greater than that required for the slab in an Interior Strip.

The concrete cover on the top bars is 2 1/2 inches, which includes a 1/2 inch wearing surface. The bottom bar cover is 1 1/2 inches. (See 18.4.6)

E18-1.11.1 Positive Moment Reinforcement for Span 1

Examine the 0.4 point of span 1

E18-1.11.1.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.5, the largest live load moment is from (LL#1), therefore at (0.4 pt.) of span 1:

M_{DC} = 22.2 kip-ft M_{DW} = 1.5 kip-ft M_{LL+IM} = 7.1 + 42.3 = 49.4 kip-ft

$M_u := 1.25\cdot(22.2) + 1.50\cdot(1.5) + 1.75\cdot(49.4)$ $\boxed{M_u = 116.5}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 583}$ psi $\boxed{\rho = 0.0108}$ $\boxed{A_s = 1.93}$ $\frac{in^2}{ft}$

For Span 1 & 3:

A_s (req'd) = 1.93 $\frac{in^2}{ft}$ (to satisfy Exterior Strip requirements)

A_s (prov'd) = 1.71 $\frac{in^2}{ft}$ (#9 at 7" c-c spacing) (to satisfy Interior Strip requirements)

Therefore, use: #9 at 6" c-c spacing (A_s = 2.00 $\frac{in^2}{ft}$) in Exterior Strip width of 5.3 ft.

E18-1.11.1.2 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.1.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

E18-1.11.2 Positive Moment Reinforcement for Span 2

Examine the 0.5 point of span 2

E18-1.11.2.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.5, the largest live load moment is from (LL#1), therefore at (0.5 pt.) of span 2:

M_{DC} = 24.0 kip-ft M_{DW} = 1.6 kip-ft M_{LL+IM} = 7.4 + 42.2 = 49.6 kip-ft

$M_u := 1.25\cdot(24.0) + 1.50\cdot(1.6) + 1.75\cdot(49.6)$ $\boxed{M_u = 119.2}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 14.9}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 597}$ psi $\qquad \boxed{\rho = 0.011} \qquad \boxed{A_s = 1.97}\ \frac{in^2}{ft}$

For Span 2:

A_s (req'd) = 1.97 $\frac{in^2}{ft}$ (to satisfy Exterior Strip requirements)

A_s (prov'd) = 2.00 $\frac{in^2}{ft}$ (#9 at 6" c-c spacing) (to satisfy Interior Strip requirements)

Therefore, use: #9 at 6" c-c spacing (A_s = 2.00 $\frac{in^2}{ft}$) in both Interior and Exterior Strips.

E18-1.11.2.2 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.2.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

E18-1.11.3 Negative Moment Reinforcement at Piers

Examine at C/L of Pier

E18-1.11.3.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The negative live load moment shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.5, the largest live load moment is from (LL#2) and therefore at (C/L of Pier) :

M_{DC} = -72.6 kip-ft $\qquad M_{DW}$ = -4.9 kip-ft $\qquad M_{LL+IM}$ = -14.0 + (-45.0) = -59.0 kip-ft

$M_u := 1.25\cdot(-72.6) + 1.50\cdot(-4.9) + 1.75\cdot(-59.0) \qquad \boxed{M_u = -201.3}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 25.5}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 344}$ psi $\boxed{\rho = 0.0061}$ $\boxed{A_s = 1.87}$ $\frac{in^2}{ft}$

At C/L Pier:

A_s (req'd) = 1.87 $\frac{in^2}{ft}$ (to satisfy Exterior Strip requirements)

A_s (prov'd) = 1.88 $\frac{in^2}{ft}$ (#8 at 5" c-c spacing) (to satisfy Interior Strip requirements)

Therefore, use: #8 at 5" c-c spacing (A_s = 1.88 $\frac{in^2}{ft}$) in both Interior and Exterior Strips

E18-1.11.3.2 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.3.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be O.K.

Edge Beam Reinforcement:

The only location where Interior Strip reinforcement is not also placed in the Exterior Strip is in Span 1 and 3 for the bottom bars, as shown in Figure E18.13.

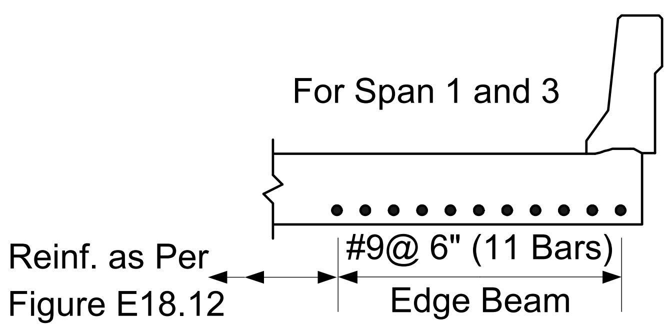

Figure E18.13

Exterior Strip Reinforcement

E18-1.11.4 Bar Steel Cutoffs

Select longitudinal reinforcement cutoff locations for an Exterior Strip.

Follow the procedure in E18-1.7.5, using reinforcement placed in the Exterior Strip. The cutoff locations must meet crack control requirements (fatigue criteria is not applied to an Exterior Strip).

E18-1.11.4.1 Span 1 Positive Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 10.5 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 10.0 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all positive reinforcement in the span at 10.0 (ft) from the C/L of pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.1.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.4.2 Span 2 Positive Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 11.0 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 11.5 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all positive reinforcement in the span at 11.0 (ft) from the C/L of each pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.2.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.4.3 Span 1 Negative Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 15.5 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 14.5 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all negative reinforcement in the span at 15.5 (ft) from the C/L of pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.3.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.11.4.4 Span 2 Negative Moment Reinforcement (Cutoffs)

It was found that 1/2 of the bars in the Exterior Strip may be cut at 13.5 (ft) from the C/L of pier. The bars in the Interior Strip are to be cut at 13.0 (ft) from the C/L of pier. In order to use the same bar mark for this reinforcement, cut 1/2 of all negative reinforcement in the span at 13.5

(ft) from the C/L of pier, if crack control criteria in Exterior Strip is satisfied. The remaining bars follow the layout as shown in Figure E18.12.

E18-1.11.4.4.1 Check Crack Control

Following the procedure in E18-1.7.1.3, the crack control check was found to be O.K.

E18-1.12 Transverse Distribution Reinforcement

The criteria for main reinforcement parallel to traffic is applied. The amount of transverse distribution reinforcement (located in bottom of slab) is to be determined as a percentage of the main reinforcing steel required for positive moment **LRFD [5.14.4.1]**.

Spans 1 & 3:

$$\text{Percentage} = \frac{100\%}{\sqrt{L}} \le 50\% \ \text{Max.} \quad \text{(L is the span length in feet)}$$

Main positive reinforcement equals #9 at 7" c-c spacing $\left(A_s = 1.7\ \frac{in^2}{ft}\right)$

$$\text{Percentage} = \frac{100\%}{\sqrt{38}} = 16.2\% < 50\% \ \text{Max.}$$

$$A_s := 0.162 \cdot (1.71) \qquad \boxed{A_s = 0.28}\ \frac{in^2}{ft}$$

Therefore, use #5 at 12" c-c spacing $A_s = 0.31\ \frac{in^2}{ft}$

Span 2:

Main positive reinforcement equals #9 at 6" c-c spacing $\left(A_s = 2.0\ \frac{in^2}{ft}\right)$

$$\text{Percentage} = \frac{100\%}{\sqrt{51}} = 14.0\% < 50\% \ \text{Max.}$$

$$A_s := 0.140 \cdot (2.00) \qquad \boxed{A_s = 0.28}\ \frac{in^2}{ft}$$

Therefore, use #5 at 12" c-c spacing $A_s = 0.31\ \frac{in^2}{ft}$

Refer to Standard 18.01 for placement of distribution reinforcement. For simplicity, the distribution reinforcement has been placed as shown in Figure E18.12.

E18-1.13 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceeding sections.

E18-1.13.1 Longitudinal and Transverse Distribution Reinforcement

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8.2]**

$$A_s \ge \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \qquad \text{and} \qquad 0.11 \le A_s \le 0.60$$

Where:

A_s = area of reinforcement in each direction and each face $\left(\frac{in^2}{ft}\right)$

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars (ksi) $\le$ 75 ksi

For cross-section of slab away from the haunch, the slab depth is 17 in., therefore:

$b := slab_{width}$ $\boxed{b = 510}$ in

$h := d_{slab}$ $\boxed{h = 17}$ in

$\boxed{f_y = 60}$ ksi

For each face, req'd A_s is:

$$A_s \ge \frac{1.30 \cdot (510) \cdot 17}{2 \cdot (510 + 17) \cdot 60} = 0.178 \ \frac{in^2}{ft}$$

, therefore, $0.11 \le A_s \le 0.60$ O.K.

For cross-section of slab at C/L of pier, the slab depth is 28 in., therefore:

$b := slab_{width}$ $\boxed{b = 510}$ in

$h := D_{haunch}$ $\boxed{h = 28}$ in

$\boxed{f_y = 60}$ ksi

For each face, req'd A_s is:

$$A_s \ge \frac{1.30 \cdot (510) \cdot 28}{2 \cdot (510 + 28) \cdot 60} = 0.288 \ \frac{in^2}{ft}$$

, therefore, $0.11 \le A_s \le 0.60$ O.K.

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches.

Max. spa = 3.0(17) = 51 in. or 18 in. governs

In **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars is also 18 inches.

All longitudinal reinforcement (top/bottom) and transverse distribution reinforcement (bottom) in the slab exceeds A_s req'd. for each face, and does not exceed maximum spacing. O.K.

E18-1.14 Shear Check of Slab

Slab bridges designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear. O.K. per **LRFD [5.14.4.1]**

E18-1.15 Longitudinal Reinforcement Tension Check

Check the longitudinal reinforcement (in bottom of slab) located at the abutments for resistance to tension caused by shear **LRFD [5.8.3.5]**, using Strength I Limit State. Calculate shear from dead load and (HL-93) live load on interior and exterior strips. Assume a diagonal crack would start at the inside edge of the bearing area.

The concrete slab rests on an A1 (fixed) abutment, which has a width of 2.5 ft. For a 6 degree skew, the distance along the C/L of the bridge is 2.52 ft. Determine the distance D_{crack} from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement.

Assume the crack angle is: $\theta := 35$ degrees

The distance from the bottom of slab to the center of tensile reinforcement is 2.06 inches.

$$D_{crack} := (2.52) + \left(\frac{2.06}{12}\right)\cdot\frac{\cot(\theta)}{\cos(6)} \qquad \boxed{D_{crack} = 2.78} \text{ ft}$$

For an interior strip:

The longitudinal reinforcement provided is #9 at 7" c-c spacing ($1.71 \frac{in^2}{ft}$)

The development length (ℓ_d) from (Table 9.9-2, Chapter 9) is 3'-9" (3.75 ft.)

The nominal tensile resistance (T_{nom}), of the longitudinal bars at the crack location is:

$$T_{nom} = A_s\cdot(f_y)\cdot\left[\frac{D_{crack} - (end\_cover)}{dev\_length}\right] \le A_s\cdot(f_y) = 102.6 \text{ kips}$$

$$T_{nom} := (1.71)\cdot 60.0\cdot\left(\frac{2.78\cdot 12 - 2}{3.75\cdot 12}\right) \qquad \boxed{T_{nom} = 71.5} \text{ kips}$$

The factored tension force (T_{fact}), from shear, to be resisted is from **LRFD [Eq'n. 5.8.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = \left(\frac{V_u}{\phi_v}\right)\cdot\cot(\theta)$$

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCmax} := 1.25$ $\gamma_{DWmax} := 1.50$ $\gamma_{LLstr1} := 1.75$ $\phi_v := 0.9$

$Q_i = V_{DC}, V_{DW}, V_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; shear due to applied loads as stated in E18-1.2

$Q = V_u = \eta_i\,[\gamma_{DCmax}(V_{DC}) + \gamma_{DWmax}(V_{DW}) + \gamma_{LLstr1}(V_{LL+IM})]$

$= 1.0\,[1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM})]$

Therefore:

$V_u = 1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM})$ (Factored Load Equation)

The live load shear shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From the computer analysis, for a one foot design width:

$V_{DC} = 2.96$ kip $V_{DW} = 0.3$ kip $V_{LL+IM} = 0.94 + 5.68 = 6.62$ kip (LL#2)

$V_u := 1.25\cdot(2.96) + 1.50\cdot(0.3) + 1.75\cdot(6.62)$ $\boxed{V_u = 15.74}$ kips (at C/L abutment)

$T_{fact} := \left(\dfrac{V_u}{\phi_v}\right)\cdot\cot(\theta)$ $\boxed{T_{fact} = 24.97}$ kips

Therefore: $T_{fact} = 24.97$ kips < $T_{nom} = 71.5$ kips O.K.

For simplicity, the value of V_u at the abutment centerline was used.

If the values for T_{fact} and T_{nom} were close, the procedure for determining the crack angle (θ) as outlined in **LRFD [5.8.3.4.2]** should be used.

The Exterior Strip was also examined and the longitudinal reinforcement was found to be satisfactory. O.K.

E18-1.16 Transverse Reinforcement in Slab over the Piers

The bridge in this example has a pier with (4) circular columns and a (2.5 ft x 2.5 ft) pier cap with rounded cap ends. (See Figure E18.14)

Out to out width of slab = $slab_{width}$ $\boxed{slab_{width} = 42.5}$ ft

Width of slab along skew = $slab_{skew} = \dfrac{42.5}{\cos(6deg)}$ $\boxed{slab_{skew} = 42.73}$ ft

Using a 6 inch offset from edge of slab to edge of pier cap per Standard 18.02 gives:

Length of Pier cap = cap_{length}

$$cap_{length} = 42.73 - 2\cdot\left(\frac{1.25+0.5}{\cos(6deg)} - 1.25\right) \qquad \boxed{cap_{length} = 41.71} \ \text{ft}$$

E18-1.16.1 Dead Load Moments

Find the reaction, S_{DL} , (on a one foot slab width) at the pier due to (DC_{slab}) and ($DC_{1/2"WS}$). This dead load will be carried by the pier cap.

From the computer analysis, $S_{DL} := 12.4 \frac{kip}{ft}$ at the pier.

For a 2.5 ft by 2.5 ft pier cap: $Cap_{DL} := 1.0 \frac{kip}{ft}$

Therefore, the uniform dead load on the pier cap = PDL

$$PDL := S_{DL} + Cap_{DL} \qquad \boxed{PDL = 13.4} \ \frac{kip}{ft}$$

Calculate the dead load moments at columns (A,B,C & D), as shown in Figure E18.14, using the three-moment equation. The moments at columns (A & D) are equal, therefore:

$$M_A = \frac{1}{2}(PDL)\cdot L^2 \qquad M_A := \frac{1}{2}\cdot(13.4)\cdot 1.25^2 \qquad \boxed{M_A = 10.5} \ \text{kip-ft} \qquad \boxed{M_D = 10.5} \ \text{kip-ft}$$

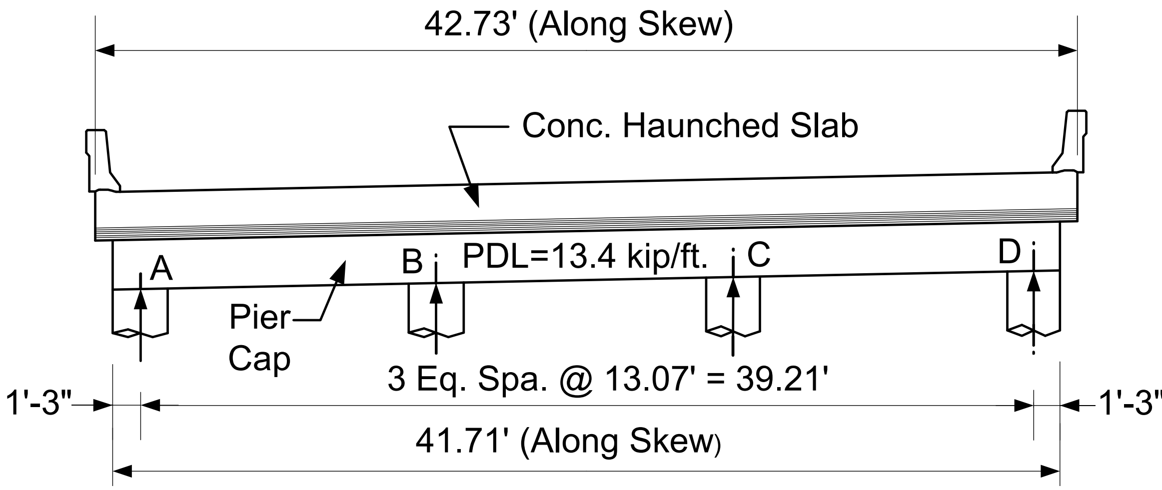

Figure E18.14

Section along C/L of Pier

Applying the three-moment equation for M_B gives values of:

$$\frac{6 \cdot A \cdot a}{L} = \frac{6 \cdot A \cdot b}{L} = \frac{(PDL) \cdot L^3}{4} \qquad \frac{(PDL) \cdot L^3}{4} = \frac{(13.4) \cdot 13.07^3}{4} = 7480 \text{ kip-ft}$$

The three-moment equation is: $M_A \cdot L_1 + 2 \cdot M_B \cdot (L_1 + L_2) + M_c \cdot L_2 + 6 \cdot \frac{A_1 \cdot a_1}{L_1} + 6 \frac{A_2 \cdot b_2}{L_2} = 0$

Refer to "Strength of Materials" textbook for derivation of the three-moment equation.

Other methods such as influence tables or moment distribution can also be used to obtain the dead load moments.

If M_A is known and due to symmetry $M_B = M_C$; the above equation reduces to one unknown, M_B, as follows:

$$(-10.5) \cdot 13.07 + 2 \cdot M_B \cdot (13.07 + 13.07) + M_B \cdot (13.07) + 7480 + 7480 = 0$$

Therefore, solving for M_B and knowing $M_C = M_B$: $\boxed{M_B = 226.8}$ kip-ft $\boxed{M_C = 226.8}$ kip-ft

Find the reaction (on a one foot slab width) at the pier due to (DC_{FWS}) and (DC_{para}). This dead load will be carried by the pier cap and a transverse beam represented by a portion of the slab over the pier.

From the computer analysis, FWS + para. (DL) = 1.9 $\frac{kip}{ft}$ at the pier.

Using the three-moment equation, $\boxed{M_A = 1.5}$ kip-ft $\boxed{M_D = 1.5}$ kip-ft

$\boxed{M_B = 32.2}$ kip-ft $\boxed{M_C = 32.2}$ kip-ft

The partial dead load moment diagram for "PDL" and "FWS + para (DL)" is shown in Figure E18.15.

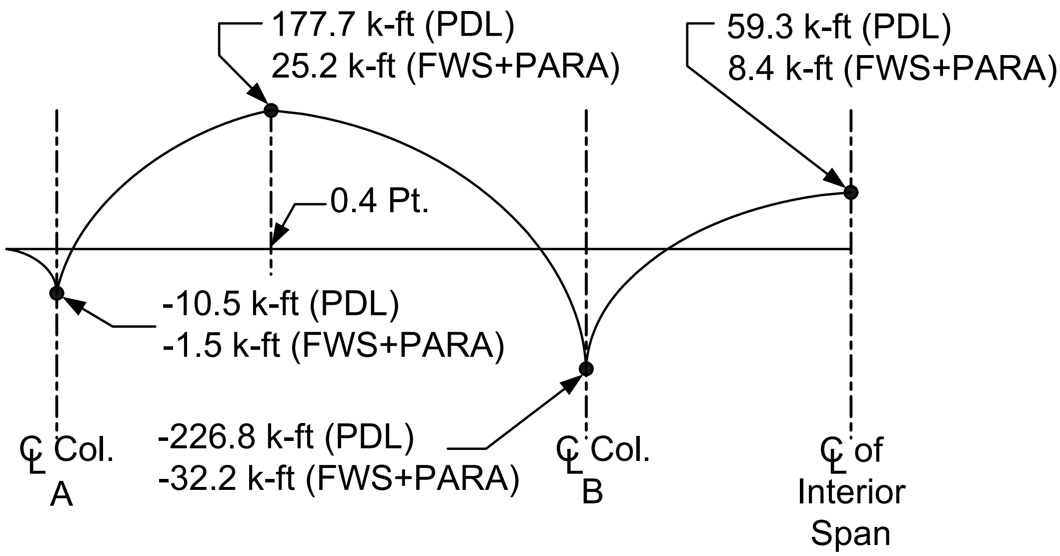

Figure E18.15

Dead Load Moment Diagram

E18-1.16.2 Live Load Moments

The maximum live load reactions at the pier shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

The reactions per lane (from computer analysis), before applying the dynamic load allowance (IM) are:

Design Lane Load = <u>35.1 kips</u>	90% Double Design Trucks = 62.1 kips
Design Tandem = 50.0 kips	90% Design Lane Load = 31.6 kips
Design Truck = <u>68.9 kips</u>	

The largest live load reaction is from: Design Truck + Design Lane Load (LL#2)

The dynamic load allowance (IM) is 33% .

Design Truck Reaction (including IM = 33%):

$$1.33\cdot(68.9) = 91.64\ \frac{\text{kip}}{\text{truck}}\text{, therefore, Wheel Load} = \frac{91.64}{2} = 45.8\ \frac{\text{kip}}{\text{wheel}}$$

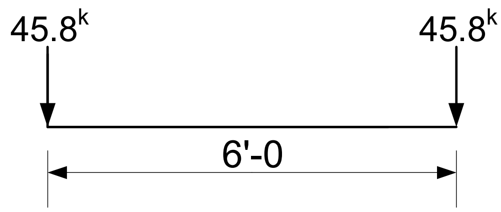

Figure E18.16

Design Truck Reaction

Design Lane Load Reaction (IM not applied to Lane Load):

$$\frac{(35.1)\text{kip}}{(10)\_\text{ft lane}} = 3.51\frac{\text{kip}}{\text{ft}}$$

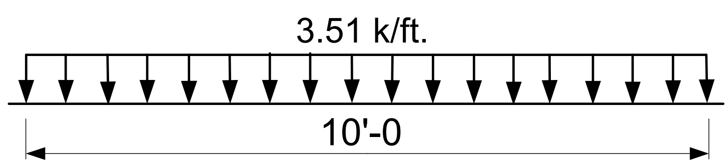

Figure E18.17

Design Lane Load Reaction

This live load is carried by the pier cap and a transverse beam represented by a portion of the slab over the pier.

Using influence lines for a 3-span continuous beam, the following results are obtained. The multiple presence factor (m) is 1.0 for (2) loaded lanes. **LRFD [3.6.1.1.2]**.

Calculate the positive live load moment, M_{LL+IM}, at (0.4 pt.) of Exterior Span

Because lane width of (10 ft) is almost equal to the span length (13.07 ft), for simplicity place uniform lane load reaction across the entire span, as shown in Figure E18.18.

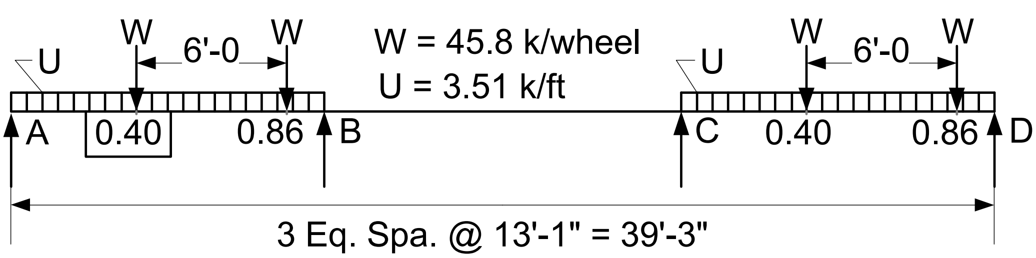

Figure E18.18

Live Load Placement for $+M_{LL+IM}$

M_{LL+IM} = (0.2042 + 0.0328 + 0.0102 + 0.0036)(45.8)(13.07) + (0.100)(3.51)(13.07)^2
= 150.1 + 60.0
= 210.1 kip-ft (Max + M_{LL+IM} in Ext. Span - 0.4 pt.)

Calculate the negative live load moment, M_{LL+IM}, at C/L of column B

Because lane width of (10 ft) is almost equal to the span length (13.07 ft), for simplicity place uniform lane load reaction across the entire span, as shown in Figure E18.19.

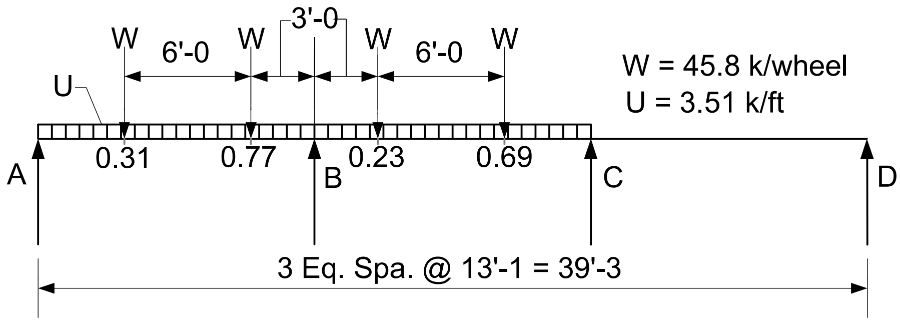

Figure E18.19

Live Load Placement for $-M_{LL+IM}$

M_{LL+IM} = (0.07448 + 0.08232 + 0.0679 + 0.0505)(45.8)(13.07) + (0.1167)(3.51)(13.07)^2
= 164.7 + 70.0
= 234.7 kip-ft (Max - M_{LL+IM} at C/L of column B)

It is assumed for this example that adequate shear transfer has been achieved **LRFD [5.8.4]** between transverse slab member and pier cap and that they will perform as a unit. Therefore, "FWS + para (DL)" and "LL + IM" will be acting on a member made up of the pier cap and the transverse slab member. Designer must insure adequate transfer if using this approach.

Calculate section width, b_{pos}, and effective depth, d_{pos} , in positive moment region, for the pier cap and the transverse slab member acting as a unit (See Figure E18.20):

b_{pos} = width of slab section = 1/2 center to center column spacing or 8 feet, whichever is smaller (See 18.4.7.2).

(C/L - C/L) column spacing x (1/2) = 6.5 ft < 8.0 ft $\boxed{b_{pos} = 78}$ in

d_{pos} = D_{haunch} + cap depth - bott. clr. - stirrup dia. - 1/2 bar dia.

$d_{pos} := 28 + 30 - 1.5 - 0.625 - 0.44$ $\boxed{d_{pos} = 55.44}$ in

Calculate section width, b_{neg}, and effective depth, d_{neg} , in negative moment region, for the pier cap and the transverse slab member acting as a unit (See Figure E18.20):

b_{neg} = width of pier cap = 2.5 ft $\boxed{b_{neg} = 30}$ in

d_{neg} = D_{haunch} + cap depth - top clr. - top bar dia. - 1/2 bar dia.

$d_{neg} := 28 + 30 - 2 - 1 - 0.38$ $\boxed{d_{neg} = 54.62}$ in

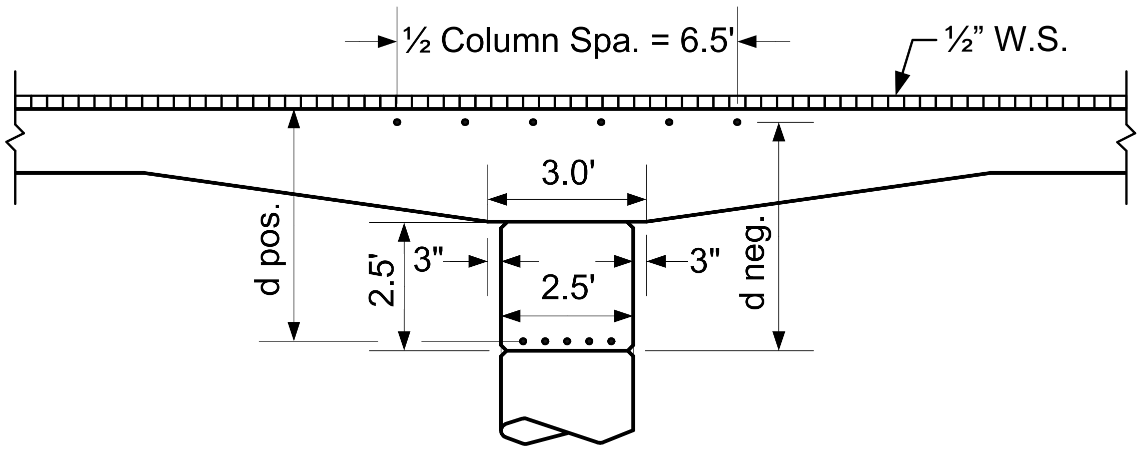

Figure E18.20

Details for Transverse Slab Member

E18-1.16.3 Positive Moment Reinforcement for Pier Cap

Examine the 0.4 point of the Exterior span

E18-1.16.3.1 Design for Strength

The dead load, PDL, carried by the pier cap is from (DC_{slab}) + ($DC_{1/2"WS}$) + Pier Cap DL.

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})$

M_{DC} = 177.7 kip-ft (See Figure E18.15)

$M_u := 1.25\cdot(177.7)$ (contribution from PDL) $\boxed{M_u = 222.1}$ kip-ft

b_{cap} = 2.5ft (pier cap width) $\boxed{b_{cap} = 30}$ in

d_s = pier cap depth - bott. clr. - stirrup dia. - 1/2 bar dia.

$d_s := 30 - 1.5 - 0.625 - 0.44$ $\boxed{d_s = 27.43}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_{s1}, are:

$\boxed{R_u = 131}$ psi $\boxed{\rho = 0.00223}$ $\boxed{A_{s1} = 1.84}$ in^2

The dead loads (FWS + para DL) and live load (LL+IM) are carried by the pier cap and the transverse slab member acting as a unit.

Split the (FWS + para DL) dead load moment (from Figure E18.15) into components:

$M_{DC} := 11.9$ kip-ft (moment from para DL)

$M_{DW} := 13.3$ kip-ft (moment from FWS)

$M_{LL+IM} = 210.1$ kip-ft

$M_u := 1.25 \cdot (11.9) + 1.50 \cdot (13.3) + 1.75 \cdot (210.1)$ $\boxed{M_u = 402.5}$ kip-ft

$\boxed{b_{pos} = 78}$ in (See E18-1.16.2)

$\boxed{d_{pos} = 55.44}$ in (See E18-1.16.2)

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_{s2}, are:

$\boxed{R_u = 22.4}$ psi $\boxed{\rho = 0.00037}$ $\boxed{A_{s2} = 1.6}$ in^2

$A_{s\_total} := A_{s1} + A_{s2}$ $\boxed{A_{s\_total} = 3.44}$ in^2

E18-1.16.4 Negative Moment Reinforcement for Pier Cap

<u>Examine at C/L of Column "B"</u>

E18-1.16.4.1 Design for Strength

The dead load, PDL, carried by the pier cap is from (DC_{slab}) + ($DC_{1/2"WS}$) + Pier Cap DL.

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})$

$M_{DC} = 226.8$ kip-ft (See Figure E18.15)

$M_u := 1.25 \cdot (226.8)$ (contribution from PDL) $\boxed{M_u = 283.5}$ kip-ft

$\boxed{b_{cap} = 30}$ in (pier cap width)

d_s = pier cap depth - top clr. - stirrup dia. - 1/2 bar dia.

$d_s := 30 - 1.5 - 0.625 - 0.44$ $\boxed{d_s = 27.43}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 167}$ psi $\boxed{\rho = 0.00286}$ $\boxed{A_s = 2.35}$ in^2

E18-1.16.5 Positive Moment Reinforcement for Transverse Slab Member

See Standard 18.01 for minimum reinforcement at this location

E18-1.16.6 Negative Moment Reinforcement for Transverse Slab Member

Examine at C/L of Column "B"

E18-1.16.6.1 Design for Strength

The dead loads (FWS + para DL) and live load (LL+IM) are carried by the pier cap and the transverse slab member acting as a unit.

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})$$

Split the (FWS + para DL) dead load moment (from Figure E18.15) into components:

$M_{DC} := 15.2$ kip-ft (moment from para DL)

$M_{DW} := 17.0$ kip-ft (moment from FWS)

$M_{LL+IM} = 234.7$ kip-ft

$M_u := 1.25 \cdot (15.2) + 1.50 \cdot (17.0) + 1.75 \cdot (234.7)$ $\boxed{M_u = 455.2}$ kip-ft

$\boxed{b_{neg} = 30}$ in (See E18-1.16.2)

$\boxed{d_{neg} = 54.62}$ in (See E18-1.16.2)

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 67.8}$ psi $\boxed{\rho = 0.00114}$ $\boxed{A_s = 1.87}$ in^2

In E18-1.16.8, check to see if this bar area meets the minimum reinforcement criteria. Then the bar size and spacing can be selected.

E18-1.16.7 Shear Check of Slab at the Pier

Check the shear (reaction) in the slab at the pier, using Strength I Limit State.

Due to the geometry and loading, stirrups are generally not required or recommended.

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCmax} := 1.25$ $\gamma_{DWmax} := 1.50$ $\gamma_{LLstr1} := 1.75$ $\phi_v := 0.9$

$Q_i = V_{DC}, V_{DW}, V_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; shear (reactions) due to applied loads as stated in E18-1.2

$Q = V_u = \eta_i\,[\gamma_{DCmax}(V_{DC}) + \gamma_{DWmax}(V_{DW}) + \gamma_{LLstr1}(V_{LL+IM})]$

$= 1.0\,[1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM})]$

$V_r = \phi_v \cdot V_n$

Therefore: $V_u \leq V_r$ (Limit States Equation)

$V_u = 1.25(V_{DC}) + 1.50(V_{DW}) + 1.75(V_{LL+IM}) \leq \phi_v\, V_n = V_r$

Find the dead load reactions at the Pier:

From the computer analysis, for a one foot design width:

V_{DC1} = reaction from $(DC_{slab}) + (DC_{1/2''WS})$ = 12.4 kip/ft

V_{DC2} = reaction from (DC_{para}) = 0.9 kip/ft

Therefore, total reaction (V_{DC}) from these loads across the slab width is:

$V_{DC} := (12.4 + 0.9)\cdot 42.5$ $\boxed{V_{DC} = 565.3}$ kips

V_{DW} = reaction from (DW_{FWS}) future wearing surface = 1.0 kip/ft

Therefore, total reaction (V_{DW}) from this load across the slab width is:

$V_{DW} := 1.0\cdot(42.5)$ $\boxed{V_{DW} = 42.5}$ kips

Find the live load reaction at the Pier:

For live load, use (3) design lanes **LRFD [3.6.1.1.1]** and multiple presence factor (m = 0.85) **LRFD [3.6.1.1.2]**.

From E18-1.16.2: Design Truck Reaction= 91.64 $\frac{kip}{truck}$ (for one lane)

Design Lane Load Reaction= 35.1 $\frac{kip}{lane}$ (for one lane)

Therefore, total reaction (V_{LL+IM}) from these loads is:

V_{LL+IM} = (91.64 + 35.1)(3 design lanes)(0.85) $\boxed{V_{LL+IM} = 323.2}$ kips

$V_u := 1.25\cdot(565.3) + 1.50\cdot(42.5) + 1.75\cdot(323.2)$ $\boxed{V_u = 1336}$ kips

Check for shear (two-way action): **LRFD [5.13.3.6.3]**

$$V_r = \phi_v \cdot V_n = \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \lambda \sqrt{f'_c} \cdot (b_o) \cdot (d_v) \le \phi_v \cdot (0.126) \cdot \lambda \sqrt{f'_c} \cdot (b_o) \cdot (d_v)$$

Where:

β_c = ratio of long side to short side of the rectangle through which reaction force is transmitted $\approx$ 41.71 ft. / 2.5 ft. = 16.7

d_v = effective shear depth = dist. between resultant tensile & compressive forces $\approx$ 24 in.

b_o = perimeter of the critical section $\approx$ 1109 in.

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Therefore, $V_r := \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v$ $\boxed{V_r = 3380}$ kips

but $\le \phi_v \cdot 0.126 \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v = 6036$ kips

Therefore, V_u = 1336 kips < V_r = 3380 kips O.K.

Note: Shear check and shear reinforcement design for the pier cap is not shown in this example. Also crack control criteria, minimum reinforcement checks, and shrinkage and temperature reinforcement checks are not shown for the pier cap.

E18-1.16.8 Minimum Reinforcement Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column) for minimum reinforcement criteria.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD [5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$f_r = 0.24\lambda\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.48}$ ksi

h = pier cap depth + D_{haunch} (section depth) $\boxed{h = 58}$ in

b_{cap} = pier cap width $\boxed{b_{cap} = 30}$ in

$I_g := \frac{1}{12} \cdot b_{cap} \cdot h^3$ (gross moment of inertia) $\boxed{I_g = 487780}$ in^4

$c := \frac{h}{2}$ (section depth/2) $\boxed{c = 29}$ in

$$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (487780)}{29(12)}$$ $\boxed{M_{cr} = 740.1}$ kip-ft

$1.33 \cdot M_u = 605.4$ kip-ft , where M_u was calculated for Strength Design in E18-1.16.6.1 and (M_u = 455.2 kip-ft)

1.33 M_u controls because it is less than M_{cr}

Recalculating requirements for (New moment = $1.33 \cdot M_u$ = 605.4 kip-ft)

$\boxed{b_{neg} = 30}$ in (See E18-1.16.2)

$\boxed{d_{neg} = 54.62}$ in (See E18-1.16.2)

Calculate R_u, coefficient of resistance:

$$R_u = \frac{M_u}{\phi_f \cdot (b_{neg}) \cdot d_{neg}^2} \qquad R_u := \frac{605.4 \cdot (12) \cdot 1000}{0.9(30) \cdot 54.62^2}$$ $\boxed{R_u = 90.2}$ psi

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.00152$

$A_s = \rho \cdot (b_{neg}) \cdot d_{neg}$ $\quad A_s := 0.00152 \cdot (30) 54.62$ $\boxed{A_s = 2.49}$ in^2

Place this reinforcement in a width, centered over the pier, equal to 1/2 the center to center column spacing or 8 feet, whichever is smaller. Therefore, width equals 6.5 feet.

Therefore, 2.49 in^2/6.5 ft. = 0.38 in^2/ft. Try <u>#5 at 9" c-c spacing</u> for a 6.5 ft. transverse width over the pier. This will provide (A_s = 2.79 in^2) in a 6.5 ft. width.

Calculate the depth of the compressive stress block

Assume $f_s = f_y$ (See 18.3.3.2.1) ; for f'_c = 4.0 ksi : $\alpha_1 := 0.85$ and $\beta_1 = 0.85$

$$a = \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot b_{neg}} \qquad a := \frac{2.79 \cdot (60)}{0.85 \cdot (4.0) \cdot 30}$$ $\boxed{a = 1.64}$ in

If $\frac{c}{d_s} \le 0.6$ for (f_y = 60 ksi) **LRFD [5.7.2.1]**, then reinforcement has yielded and the assumption is correct.

$\beta_1 := 0.85$ $\quad c := \frac{a}{\beta_1}$ $\quad \boxed{c = 1.93}$ in

$d_s := d_{neg}$ $\quad \boxed{d_s = 54.62}$ in

$\frac{c}{d_s} = 0.04 < 0.6$ therefore, the reinforcement will yield.

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

$$M_r := 0.9 \cdot (2.79) \cdot 60.0 \cdot \left(\frac{54.62 - \frac{1.64}{2}}{12}\right) \qquad \boxed{M_r = 675.5} \text{ kip-ft}$$

Therefore, $1.33(M_u)$ = 605.4 kip-ft < M_r = 675.5 kip-ft O.K.

E18-1.16.9 Crack Control Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column).

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\quad \boxed{f_{r80\%} = 0.38}$ ksi $\quad \boxed{c = 29}$ in $\quad \boxed{I_g = 487780}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Using same moments selected for Strength Design in E18-1.16.6, at (interior column), provides

M_{DC} = 15.2 kip-ft $\quad M_{DW}$ = 17.0 kip-ft $\quad M_{LL+IM}$ = 234.7 kip-ft

$M_s := 1.0 \cdot (15.2) + 1.0(17.0) + 1.0 \cdot (234.7)$ $\quad \boxed{M_s = 266.9}$ kip-ft

$f_T = \frac{M_s \cdot c}{I_g}$ $\quad f_T := \frac{266.9 \cdot (29) \cdot 12}{487780}$ $\quad \boxed{f_T = 0.19}$ ksi

f_T =0.19 ksi < 80% f_r = 0.38 ksi; therefore, crack control criteria check is not req'd.

Therefore, crack control criteria for transverse slab reinforcement is O.K.

Use: #5 at 9" c-c spacing for a 6.5 ft. transverse width over the pier.

The transverse slab member reinforcement (top/bottom), and the remainder of the transverse reinforcement is shown in Figure E18.21.

E18-1.17 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the remaining transverse reinforcement.

E18-1.17.1 Transverse Slab Member and Other Transverse Reinforcement

Following the procedure in E18-1.13.1:

All transverse slab member reinforcement (top/bottom) and remainder of transverse reinforcement in slab exceeds A_s req'd. for each face, and does not exceed maximum spacing.

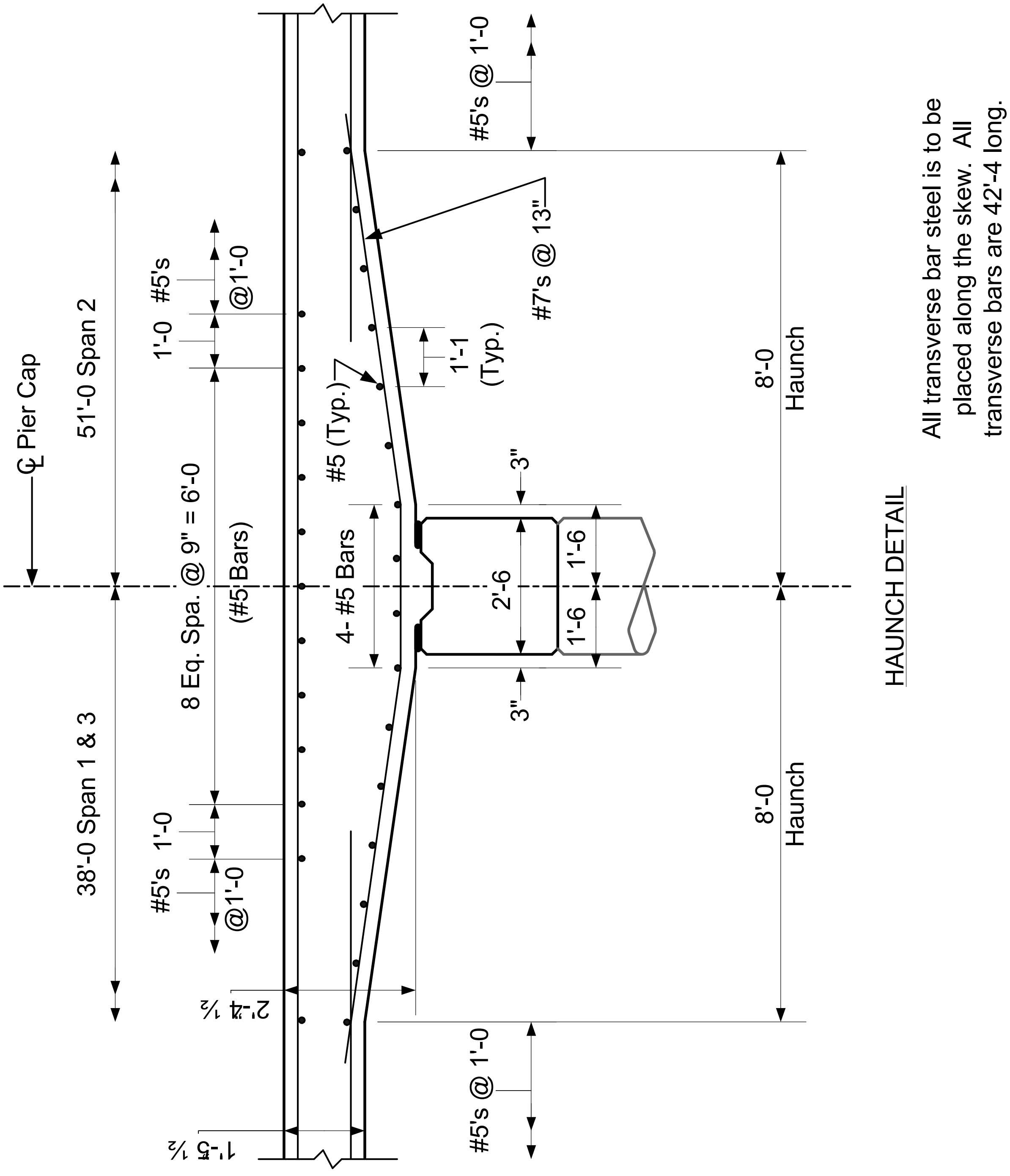

Figure E18.21

Haunch Detail

E18-1.18 Check for Uplift at Abutments

Check for uplift at the abutments, using Strength I Limit State **LRFD [C3.4.1, 5.5.4.3, 14.6.1]**

The maximum uplift at the abutments from live load is obtained from the following influence line and shall be the largest caused by live loads (LL#1 or LL#2) in each design lane (See Figure E18.22). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

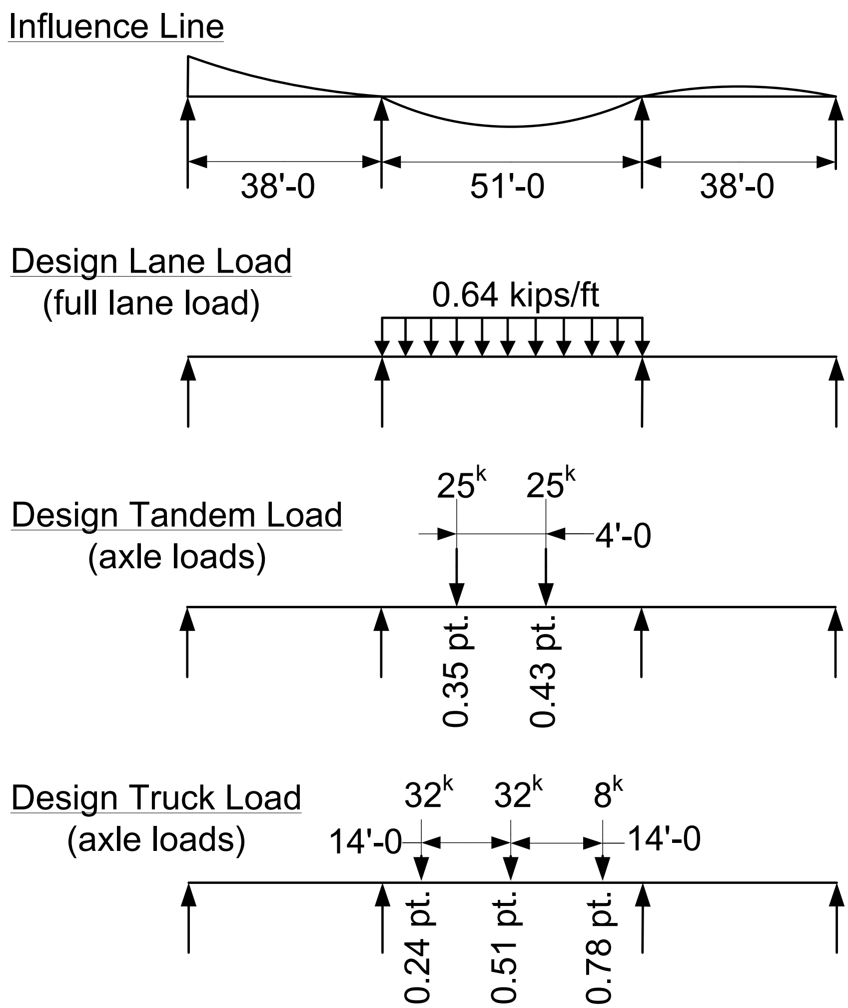

Figure E18.22

Influence Line and Live Loads for Uplift

Tables of influence line coefficients are used to calculate live load reactions at the abutment. The reactions per lane, before applying the dynamic load allowance (IM) are:

Design Lane Load Reaction = (0.1206)(0.64)(38.0) = 2.9 kips

Design Truck Reaction = (0.1290 + 0.1360)(32) + (0.060)(8) = 9.0 kips

Design Tandem Reaction = (0.149 + 0.148)(25) = 7.4 kips

The largest live load reaction is from: Design Truck + Design Lane Load (LL#2)

The dynamic load allowance (IM) is 33% ; (applied to Design Truck)

Therefore, total live load reaction (R_{LL+IM}) from these loads is:

R_{LL+IM} = 9.0(1.33) + 2.9 = 14.87 kips (for one lane)

Find the dead load reactions at the abutment:

From the computer analysis, for a <u>one foot design width</u>:

R_{DC1} = reaction from (DC_{slab}) + ($DC_{1/2"WS}$) = 2.8 kip/ft

R_{DC2} = reaction from (DC_{para}) = 0.3 kip/ft

Therefore, total dead load reaction (R_{DC}) from these loads across the slab width is:

$R_{DC} := (2.8 + 0.3) \cdot 42.5$ $\boxed{R_{DC} = 131.75}$ kips

Total dead load reaction ignores (DW_{FWS}) because it reduces uplift.

Check uplift for Strength I Limit State:

Looking at E18-1.2: $\eta_i := 1.0$ and Table E18.1: $\gamma_{DCmin} := 0.90$ $\gamma_{LLstr1} := 1.75$

Dead Load Reaction at Abutments = γ_{DCmin}(R_{DC}) = 0.90(131.75) = <u>118.6 kips</u>

Uplift from Live Load = γ_{LLstr1}(R_{LL+IM})(# lanes loaded)(m)

Use (3) design lanes **LRFD [3.6.1.1.1]** and multiple presence factor (m = 0.85) **LRFD [3.6.1.1.2]**

Uplift from Live Load = 1.75(14.87)(3 design lanes)(0.85) = <u>66.4 kips</u>

Therefore, Uplift = 66.4 kips < Dead Load Reaction = 118.6 kips <u>O.K.</u>

Because dead load reaction at abutments exceeds uplift from live load, the existing dowels (#5 at 1'-0 spa.) are adequate. (See Standard 12.01)

E18-1.19 Deflection Joints and Construction Joints

Locate deflection joints for concrete slab structures according to Standard 30.07. Refer to Standards 18.01/18.02 for recommended construction joint guidelines.

Note: See Standard 18.01/18.02 for required notes and other details

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Table of Contents

19.1 Introduction

This chapter provides information intended for prestressed I-girders. Prestressed box girders and general prestressed concrete guidelines are also included in this chapter.

The definition of prestressed concrete as given by the ACI Committee on Prestressed Concrete is:

> "Concrete in which there has been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree. In reinforced concrete members the prestress is commonly introduced by tensioning the steel reinforcement."

This internal stress is induced into the member by either of the following prestressing methods.

19.1.1 Pretensioning

In pretensioning, the tendons are first stressed to a given level and then the concrete is cast around them. The tendons may be composed of wires, bars or strands.

The most common system of pretensioning is the long line system, by which a number of units are produced at once. First the tendons are stretched between anchorage blocks at opposite ends of the long stretching bed. Next the spacers or separators are placed at the desired member intervals, and then the concrete is placed within these intervals. When the concrete has attained a sufficient strength, the steel is released and its stress is transferred to the concrete via bond.

19.1.2 Post-Tensioning

In post-tensioning, the concrete member is first cast with one or more post-tensioning ducts or tubes for future insertion of tendons. Once the concrete is sufficiently strong, the tendons are stressed by jacking against the concrete. When the desired prestress level is reached, the tendons are locked under stress by means of end anchorages or clamps. Subsequently, the duct is filled with grout to protect the steel from corrosion and give the added safeguard of bond.

In contrast to pretensioning, which is usually incorporated in precasting (casting away from final position), post-tensioning lends itself to cast-in-place construction.

19.2 Basic Principles

This section defines the internal stress that results from either prestressing method.

First consider the simple beam shown in Figure 19.2-1.

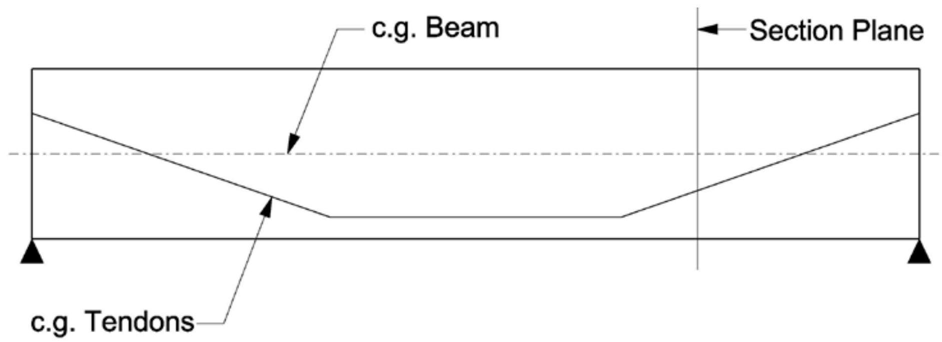


Figure 19.2-1
Simple Span Prestressed Concrete Beam

The horizontal component, P, of the tendon force, F, is assumed constant at any section along the length of the beam.

Also, at any section of the beam the forces in the beam and in the tendon are in equilibrium. Forces and moments may be equated at any section.

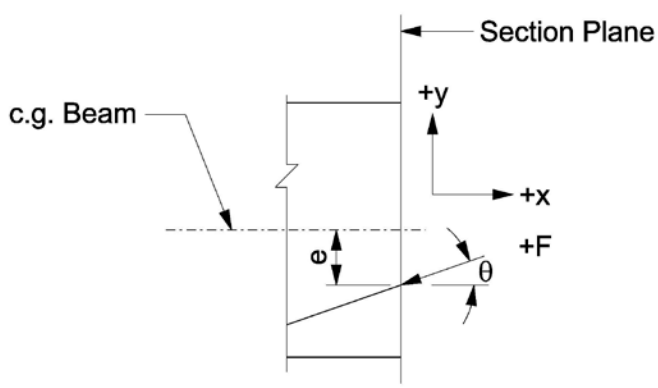


Figure 19.2-2
Assumed Sign Convention for Section Forces

The assumed sign convention is as shown in Figure 19.2-2 with the origin at the intersection of the section plane and the center of gravity (centroidal axis) of the beam. This convention indicates compression as positive and tension as negative.

The eccentricity of the tendon can be either positive or negative with respect to the center of gravity; therefore it is unsigned in the general equation. The reaction of the tendon on the beam is always negative; therefore the horizontal component is signed as:

$$P = F\cos\theta$$

Then, by equating forces in the x-direction, the reaction, P, of the tendon on the concrete produces a compressive stress equal to:

$$f_1 = \frac{P}{A}$$

Where:

A	=	Cross-sectional area of the beam

Since the line of action of the reaction, P, is eccentric to the centroidal axis of the beam by the amount e, it produces a bending moment.

$$M = Pe$$

This moment induces stresses in the beam given by the flexure formula:

$$f_2 = \frac{My}{I} = \frac{Pey}{I}$$

Where:

y	=	Distance from the centroidal axis to the fiber under consideration, with an unsigned value in the general equations
I	=	Moment of inertia of the section about its centroidal axis

The algebraic sum of f_1 and f_2 yields an expression for the total prestress on the section when the beam is not loaded.

$$f_p = f_1 + f_2 = \frac{P}{A} + \frac{Pey}{I}$$

Now, by substituting $I = Ar^2$, where r is the radius of gyration, into the above expression and arranging terms, we have:

$$f_p = \frac{P}{A}\left(1 + \frac{ey}{r^2}\right)$$

These stress conditions are shown in Figure 19.2-3.

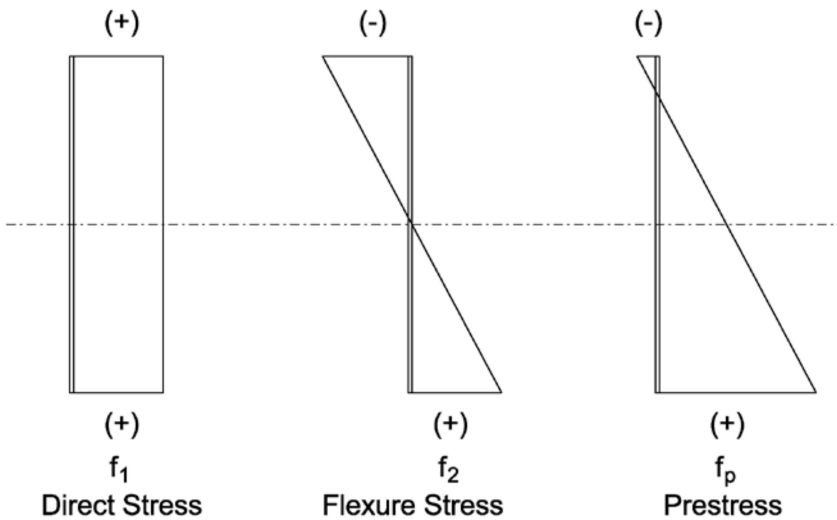


Figure 19.2-3
Calculation of Concrete Stress Due to Prestress Force

Finally, we equate forces in the y-direction which yields a shear force, V, over the section of the beam due to the component of the tendon reaction.

$$V = F\sin\theta = P\tan\theta$$

19.3 Pretensioned Member Design

This section outlines several important considerations associated with the design of conventional pretensioned members.

19.3.1 Design Strengths

The typical specified design strengths for pretensioned members are:

Prestressed I-girder concrete:	f'_c	= 6 to 8 ksi
Prestressed box girder concrete:	f'_c	= 5 ksi
Prestressed concrete (at release):	f'_{ci}	= 0.75 to 0.85 $f'_c \leq 6.8$ ksi
Deck and diaphragm concrete:	f'_c	= 4 ksi
Prestressing steel:	f_{pu}	= 270 ksi
Grade 60 reinforcement:	f_y	= 60 ksi

The *actual required* compressive strength of the concrete at prestress transfer, f'_{ci}, is to be stated on the plans. For typical prestressed girders, $f'_{ci(min)}$ is $0.75(f'_c)$.

WisDOT policy item:

For prestressed I-girders, the use of concrete with strength greater than 8 ksi is only allowed with the prior approval of the BOS Development Section. Occasional use of strengths up to 8.5 ksi may be allowed. Strengths exceeding these values are difficult for local fabricators to consistently achieve as the coarse aggregate strength becomes the controlling factor.

For prestressed box girders, the use of concrete with strength greater than 5 ksi is only allowed with prior approval of the BOS Development Section.

The use of 8 ksi concrete for prestressed I-girders and 6.8 ksi for f'_{ci} still allows the fabricator to use a 24-hour cycle for girder fabrication. There are situations in which higher strength concrete in the prestressed I-girders may be considered for economy, provided that f'_{ci} does not exceed 6.8 ksi. Higher strength concrete may be considered if the extra strength is needed to avoid using a less economical superstructure type or if a shallower girder can be provided and its use justified for sufficient reasons (min. vert. clearance, etc.) Using higher strength concrete to eliminate a girder line is not the preference of the Bureau of Structures. It is often more economical to add an extra girder line than to use debonded strands with the minimum number of girder lines. After the number of girders has been determined, adjustments in girder spacing should be investigated to see if slab thickness can be minimized and balance between interior and exterior girders optimized.

Prestressed I-girders below the required 28-day concrete strength (or 56-day concrete strength for f'_c = 8 ksi) will be accepted if they provide strength greater than required by the design and at the reduction in pay schedule in the *Wisconsin Standard Specifications for Highway and Structure Construction*.

Low relaxation prestressing strands are required.

19.3.2 Loading Stages

The loads that a member is subjected to during its design life and those stages that generally influence the design are discussed in **LRFD [5.9]** and in the following sections. The allowable stresses at different loading stages are defined in **LRFD [5.9.3]** and **LRFD [5.9.4]**.

19.3.2.1 Prestress Transfer

Prestress transfer is the initial condition of prestress that exists immediately following the release of the tendons (transfer of the tendon force to the concrete). The eccentricity of the prestress force produces an upward camber. In addition, a stress due to the dead load of the member itself is also induced. This is a stage of temporary stress that includes a reduction in prestress due to elastic shortening of the member.

19.3.2.2 Losses

After elastic shortening losses, the external loading is the same as at prestress transfer. However, the internal stress due to the prestressing force is further reduced by losses resulting from relaxation due to creep of the prestressing steel together with creep and shrinkage of the concrete. It is assumed that all losses occur prior to application of service loading.

LRFD [5.9.5] provides guidance about prestress losses for both pretensioned and post-tensioned members. This section presents a refined and approximate method for the calculation of time-dependent prestress losses such as concrete creep and shrinkage and prestressing steel relaxation.

<u>WisDOT policy item:</u>

WisDOT policy is to use the approximate method described in **LRFD [5.9.5.3]** to determine time-dependent losses, since this method does not require the designer to assume the age of the concrete at the different loading stages.

Losses for pretensioned members that are considered during design are listed in the following sections.

19.3.2.2.1 Elastic Shortening

Per **LRFD [5.9.5.2.3a]**, the loss due to elastic shortening, Δf_{pES1} (ksi), in pretensioned concrete members shall be taken as:

$$\Delta f_{pES1} = \frac{E_p}{E_{ct}} f_{cgp}$$

Where:

E_p	=	Modulus of elasticity of prestressing steel = 28,500 ksi **LRFD [5.4.4.2]**
E_{ct}	=	Modulus of elasticity of concrete at transfer or time of load application in ksi (see 19.3.3.8)
f_{gcp}	=	Concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)

19.3.2.2.2 Time-Dependent Losses

Per **LRFD [5.9.5.3],** an estimate of the long-term losses due to steel relaxation as well as concrete creep and shrinkage on standard precast, pretensioned members shall be taken as:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

Where:

$$\gamma_h = 1.7 - 0.01H$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})}$$

f_{pi}	=	Prestressing steel stress immediately prior to transfer (ksi)
H	=	Average annual ambient relative humidity in %, taken as 72% in Wisconsin
Δf_{pR}	=	Relaxation loss estimate taken as 2.4 ksi for low relaxation strands or 10.0 ksi for stress-relieved strands (ksi)

The losses due to elastic shortening must then be added to these time-dependent losses to determine the total losses. For non-standard members with unusual dimensions or built using staged segmental construction, the refined method of **LRFD [5.9.5.4]** shall be used. For prestressed box girders time-dependent losses shall be determined using the refined method of **LRFD [5.9.5.4]**.

19.3.2.2.3 Fabrication Losses

Fabrication losses are not considered by the designer, but they affect the design criteria used during design. Anchorage losses which occur during stressing and seating of the prestressed strands vary between 1% and 4%. Losses due to temperature change in the strands during cold weather prestressing are 6% for a 60°F change. The construction specifications permit a 5% difference in the jack pressure and elongation measurement without any adjustment.

19.3.2.3 Service Load

During service load, the member is subjected to the same loads that are present after prestress transfer and losses occur, in addition to the effects of the prestressed I-girder and prestressed box girder load-carrying behavior described in the next two sections.

19.3.2.3.1 Prestressed I-Girder

In the case of a prestressed I-girder, the dead load of the deck and diaphragms are always carried by the basic girder section on a simple span. At strand release, the girder dead load moments are calculated based on the full girder length. For all other loading stages, the girder dead load moments are based on the span length. This is due to the type of construction used (that is, unshored girders simply spanning from one substructure unit to another for single-span as well as multi-span structures).

The live load plus dynamic load allowance along with any superimposed dead load (curb, parapet or median strip which is placed after the deck concrete has hardened) are carried by the continuous composite section.

WisDOT exception to AASHTO:

The standard pier diaphragm is considered to satisfy the requirements of **LRFD [5.14.1.4.5]** and shall be considered to be fully effective.

In the case of multi-span structures with fully effective diaphragms, the longitudinal distribution of the live load, dynamic load allowance and superimposed dead loads are based on a continuous span structure. This continuity is achieved by:

a. Placing non-prestressed (conventional) reinforcement in the deck area over the interior supports.

b. Casting concrete between and around the abutting ends of adjacent girders to form a diaphragm at the support. Girders shall be in line at interior supports and equal numbers of girders shall be used in adjacent spans. The use of variable numbers of girders between spans requires prior approval by BOS.

If the span length ratio of two adjacent spans exceeds 1.5, the girders are designed as simple spans. In either case, the stirrup spacing is detailed the same as for continuous spans and bar steel is placed over the supports equivalent to continuous span design. It should be noted that this value of 1.5 is not an absolute structural limit.

19.3.2.3.2 Prestressed Box Girder

In the case of prestressed box girders with a thin concrete overlay, the dead load together with the live load and dynamic load allowance are carried by the basic girder section.

When this girder type has a composite section, the dead load of the deck is carried by the basic section and the live load, dynamic load allowance and any superimposed dead loads are carried by the composite section. A composite section shall consist of a reinforced deck, 6" minimum thickness, with composite shear reinforcement extending into the deck.

WisDOT policy item:

The use of prestressed box girders is subject to prior-approval by the Bureau of Structures. These structures are currently limited to the following requirements:

- Single spans
- Composite section details (design and rating based on non-composite section)
- 30 degree maximum skew
- AADT < 3,500 on non-NHS roadways

Variations to these requirements require approval by the Bureau of Structures.

19.3.2.4 Factored Flexural Resistance

At the final stage, the factored flexural resistance of the composite section is considered. Since the member is designed on a service load basis, it must be checked for its factored flexural resistance at the Strength I limit state. See section 17.2.3 for a discussion on limit states.

The need for both service load and strength computations lies with the radical change in a member's behavior when cracks form. Prior to cracking, the gross area of the member is effective. As a crack develops, all the tension in the concrete is picked up by the reinforcement. If the percentage of reinforcement is small, there is very little added capacity between cracking and failure.

19.3.2.5 Fatigue Limit State

At the final stage, the member is checked for the Fatigue I limit state. See section 17.2.3 for a discussion on limit states. Allowable compressive stresses in the concrete and tensile stresses in the non-prestressed reinforcement are checked.

19.3.3 Design Procedure

The intent of this section is to provide the designer with a general outline of steps for the design of pretensioned members. Sections of interest during design include, but are not limited to, the following locations:

- 10th points
- Hold-down points

- Regions where the prestress force changes (consider the effects of transfer and development lengths, as well as the effects of debonded strands)

- Critical section(s) for shear

The designer must consider the amount of prestress force at each design section, taking into account the transfer length and development length, if appropriate.

19.3.3.1 Prestressed I-Girder Member Spacing

A trial prestressed I-girder arrangement is made by using Table 19.3-1 and Table 19.3-2 as a guide. An ideal spacing results in equal strands for interior and exterior girders, together with an optimum slab thickness. Current practice is to use a minimum haunch of (1-1/4" plus deck cross slope times one-half top flange width) for section property calculations and then use a 3" average haunch for concrete preliminary quantity calculations. After preliminary design this value should be revised as needed as outlined in 19.3.4. The maximum slab overhang dimensions are detailed in 17.6.2.

For prestressed I-girder bridges, other than pedestrian or other unusual structures, four or more girders shall be used.

19.3.3.2 Prestressed Box Girder Member Spacing

The prestressed box girder is used in an adjacent multi-beam system only. Precast units are placed side by side and locked (post-tensioned) together. The span length, desired roadway width and live loading control the size of the member.

When selecting a 3' wide section vs. 4' wide section, do not mix 3' wide and 4' wide sections across the width of the bridge. Examine the roadway width produced by using all 3' wide sections or all 4' wide sections and choose the system that is the closest to but greater than the required roadway width. While 3' wide sections may produce a slightly narrower roadway width 4' wide sections are still preferred since they require fewer sections. Verify the required roadway width is possible when considerations are made for the roadway cross-slope. Table 19.3-3 states the approximate span limitations for each section depth. Coordinate roadway width with roadway designers and consider some variability. See the Standards for prestressed box girder details.

19.3.3.3 Dead Load

For a detailed discussion of the application of dead load, refer to 17.2.4.1.

The dead load moments and shears due to the girder and concrete deck are computed for simple spans. When superimposed dead loads are considered, the superimposed dead load moments are based on continuous spans.

A superimposed dead load of 20 psf is to be included in all designs which account for a possible future concrete overlay wearing surface. The future wearing surface shall be applied between the faces of curbs or parapets and shall be equally distributed among all the girders in the cross section.

For a cross section without a sidewalk, any curb or parapet dead load is distributed equally to all girders.

For a cross section with a sidewalk and barrier on the overhang, sidewalk and barrier dead loads shall be applied to the exterior girder by the lever rule. These loads shall also be applied to the interior girder by dividing the weight equally among all the girders. A more detailed discussion of dead load distribution can be found in 17.2.8.

19.3.3.4 Live Load

The HL-93 live load shall be used for all new bridges. Refer to section 17.2.4.2 for a detailed description of the HL-93 live load, including the design truck, design tandem, design lane, and double truck.

19.3.3.5 Live Load Distribution

The live load distribution factors shall be computed as specified in **LRFD [4.6.2.2].** Table 17.2-7 summarizes the equations required for prestressed I-girders. The moment and shear distribution factors for prestressed I-girders are determined using equations that consider girder spacing, span length, deck thickness, the number of girders, skew and the longitudinal stiffness parameter. See the WisDOT policy item for live load distribution factors for prestressed box girders.

Separate shear and moment distribution factors are computed for interior and exterior girders. The applicability ranges of the distribution factors shall also be considered. If the applicability ranges are not satisfied, then conservative assumptions must be made based on sound engineering judgment.

WisDOT policy item:

The typical cross section for prestressed box girders shall be type “g” as illustrated in **LRFD [Table 4.6.2.2.1-1]**.

For prestressed box girders, the St. Venant torsional inertia, J, may be calculated as closed thin-walled sections for sections with voids, and as solid sections for sections without voids in accordance with **LRFD [C4.6.2.2.1]**.

See 17.2.8 for additional information regarding live load distribution.

19.3.3.6 Dynamic Load Allowance

The dynamic load allowance, IM, is given by **LRFD [3.6.2]**. Dynamic load allowance equals 33% for all live load limit states except the fatigue limit state and is not applied to pedestrian loads or the lane load portion of the HL-93 live load. See 17.2.4.3 for further information regarding dynamic load allowance.

19.3.3.7 Prestressed I-Girder Deck Design

The design of concrete decks on prestressed I-girders is based on **LRFD [4.6.2.1]**. Moments from truck wheel loads are distributed over a width of deck which spans perpendicular to the girders. This width is known as the distribution width and is given by **LRFD [Table 4.6.2.1.3-1]**. See 17.5 for further information regarding deck design.

19.3.3.8 Composite Section

The effective flange width is the width of the deck slab that is to be taken as effective in composite action for determining resistance for all limit states. The effective flange width, in accordance with **LRFD [4.6.2.6]**, is equal to the tributary width of the girder for interior girders. For exterior girders, it is equal to one half the effective flange width of the adjacent interior girder plus the overhang width. The effective flange width shall be determined for both interior and exterior beams.

Since the deck concrete has a lower strength than the girder concrete, it also has a lower modulus of elasticity. Therefore, when computing composite section properties, the effective flange width (as stated above) must be reduced by the ratio of the modulus of elasticity of the deck concrete divided by the modulus of elasticity of the girder concrete.

WisDOT exception to AASHTO:

WisDOT uses the formulas shown below to determine E_c for prestressed girder design. For 6 ksi girder concrete, E_c is 5,500 ksi, and for 4 ksi deck concrete, E_c is 4,125 ksi. The E_c value of 5,500 ksi for 6 ksi girder concrete strength was determined from deflection studies. These equations are used in place of those presented in **LRFD [5.4.2.4]** for the following calculations: strength, section properties, and deflections due to externally applied dead and live loads.

For slab concrete strength other than 4 ksi, E_c is calculated from the following formula:

$$E_c = \frac{4{,}125\sqrt{f'_c}}{\sqrt{4}} \text{ (ksi)}$$

For girder concrete strengths other than 6 ksi, E_c is calculated from the following formula:

$$E_c = \frac{5{,}500\sqrt{f'_c}}{\sqrt{6}} \text{ (ksi)}$$

WisDOT policy item:

WisDOT uses the equation presented in **LRFD [5.4.2.4]** (and shown below) to calculate the modulus of elasticity at the time of release using the specified value of f'ci. This value of E_i is used for loss calculations and for girder camber due to prestress forces and girder self weight.

$$E_c = 33{,}000 \cdot K_1 \cdot w_c^{1.5}\sqrt{f'_{ci}}$$

Where:

K_1	=	Correction factor for source of aggregate, use 1.0 unless previously approved by BOS.
w_c	=	Unit weight of concrete, 0.150 (kcf)
f'_{ci}	=	Specified compressive strength of concrete at the time of release (ksi)

19.3.3.9 Design Stress

In many cases, stress at the Service III limit state in the bottom fiber at or near midspan after losses will control the flexural design. Determine a trial strand pattern for this condition and proceed with the flexural design, adjusting the strand pattern if necessary.

The design stress is the sum of the Service III limit state bottom fiber stresses due to non-composite dead load on the basic girder section, plus live load, dynamic load allowance and superimposed dead load on the composite section, as follows:

$$f_{des} = \frac{M_{d(nc)}}{S_{b(nc)}} + \frac{M_{d(c)} + M_{(LL+IM)}}{S_{b(c)}}$$

Where:

f_{des}	=	Service III design stress at section (ksi)
$M_{d(nc)}$	=	Service III non-composite dead load moment at section (k-in)
$M_{d(c)}$	=	Service III superimposed dead load moment at section (k-in)
$M_{(LL+IM)}$	=	Service III live load plus dynamic load allowance moment at section (k-in)
$S_{b(nc)}$	=	Non-composite section modulus for bottom of basic beam (in^3)
$S_{b(c)}$	=	Composite section modulus for bottom of basic beam (in^3)

The point of maximum stress is generally 0.5 of the span for both end and intermediate spans. But for longer spans (over 100'), the 0.4 point of the end span may control and should be checked.

19.3.3.10 Prestress Force

With f_{des} known, compute the required effective stress in the prestressing steel after losses, f_{pe}, needed to counteract all the design stress except an amount of tension equal to the tensile stress limit listed in **LRFD [Table 5.9.4.2.2-1]**. The top of the girder is subjected to severe corrosion conditions and the bottom of the girder is subjected to moderate exposure. The Service III tensile stress at the bottom fiber after losses for pretensioned concrete shall not exceed $0.19\lambda\sqrt{f'_c}$ (or 0.6 ksi) ; where λ = concrete density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight concrete. Therefore:

$$f_{pe} = f_{des} - \min(0.19\sqrt{f'_c} \text{ or } 0.6 \text{ ksi})$$

Note: A conservative approach used in hand calculations is to assume that the allowable tensile stress equals zero.

Applying the theory discussed in 19.2:

$$f_{pe} = \frac{P_{pe}}{A}\left(1 + \frac{ey}{r^2}\right)$$

Where:

P_{pe}	=	Effective prestress force after losses (kips)
A	=	Basic beam area (in^2)
e	=	Eccentricity of prestressing strands with respect to the centroid of the basic beam at section (in)
r	=	$\sqrt{\frac{I}{A}}$ of the basic beam (in)

For prestressed box girders, assume an e and apply this to the above equation to determine P_{pe} and the approximate number of strands. Then a trial strand pattern is established using the Standard Details as a guide, and a check is made on the assumed eccentricity. For prestressed I-girders, f_{pe} is solved for several predetermined patterns and is tabulated in the Standard Details.

Present practice is to detail all spans of equal length with the same number of strands, unless a span requires more than three additional strands. In this case, the different strand arrangements are detailed along with a plan note stating: "The manufacturer may furnish all girders with the greater number of strands."

19.3.3.11 Service Limit State

Several checks need to be performed at the service limit state. Refer to the previous narrative in 19.3.3 for sections to be investigated and section 17.2.3.2 for discussion on the service limit state. Note that Service I limit state is used when checking compressive stresses and Service III limit state is used when checking tensile stresses.

The following should be verified by the engineer:

- Verify that the Service III tensile stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed the limits presented in **LRFD [Table 5.9.4.1.2-1]**, which depend upon whether or not the strands are bonded and satisfy stress requirements. This will generally control at the top of the beam near the beam ends where the dead load moment approaches zero and is not able to counter the tensile stress at the top of the beam induced by the prestress force. When the calculated tensile stress exceeds the stress limits, the strand pattern must be modified by draping or partially debonding the strand configuration.

- Verify that the Service I compressive stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed 0.65 f'_{ci}, as presented in **LRFD [5.9.4.1.1]**. This will generally control at the bottom of the beam near the beam ends or at the hold-down point if using draped strands.

- Verify that the Service III tensile stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.4.2.2-1]**. No tensile stress shall be permitted for unbonded strands. The tensile stress of bonded strands shall not exceed $0.19\lambda\sqrt{f'_c}$ (or 0.6 ksi) as all strands shall be considered to be in moderate corrosive conditions. This will generally control at the bottom of the beam near midspan and at the top of the continuous end of the beam. The value of λ is 1.0 for normal weight concrete **LRFD [5.4.2.8]**.

- Verify that the Service I compressive stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.4.2.1-1]**. Two checks need to be made for girder bridges. The compressive stress due to the sum of effective prestress and permanent loads shall not exceed 0.45 f'_c (ksi). The compressive stress due to the sum of effective prestress, permanent loads and transient loads shall not exceed $0.60\phi_w f'_c$ (ksi). The term ϕ_w, a reduction factor applied to thin-walled box girders, shall be 1.0 for WisDOT standard girders.

- Verify that Fatigue I compressive stress due to fatigue live load and one-half the sum of effective prestress and permanent loads does not exceed 0.40 f'_c (ksi) **LRFD [5.5.3.1]**.

- Verify that the Service I compressive stress at the top of the deck due to all dead and live loads applied to the appropriate sections after losses does not exceed 0.40 f'_c.

WisDOT policy item:

The top of the prestressed I-girders at interior supports shall be designed as reinforced concrete members at the strength limit state in accordance with **LRFD [5.14.1.4.6]**. In this case, the stress limits for the service limit state shall not apply to this region of the precast girder.

19.3.3.12 Raised, Draped or Partially Debonded Strands

When straight strands are bonded for the full length of a prestressed girder, the tensile and compressive stresses near the ends of the girder will likely exceed the allowable service limit state stresses. This occurs because the strand pattern is designed for stresses at or near midspan, where the dead load moment is highest and best able to balance the effects of the prestress. Near the ends of the girder this dead load moment approaches zero and is less able to balance the prestress force. This results in tensile stresses in the top of the girder and compressive stresses in the bottom of the girder. The allowable initial tensile and compressive stresses are presented in the first two bullet points of 19.3.3.11. These stresses are a function of f'_{ci}, the compressive strength of concrete at the time of prestress force transfer. Transfer and development lengths should be considered when checking stresses near the ends of the girder.

The designer should start with a straight (raised), fully bonded strand pattern. If this overstresses the girder near the ends, the following methods shall be utilized to bring the girder within the allowable stresses. These methods are listed in order of preference and discussed in the following sections:

1. Use raised strand pattern (If excessive top flange reinforcement or if four or more additional strands versus a draped strand pattern are required, consider the draped strand alternative)

2. Use draped strand pattern

3. Use partially debonded strand pattern (to be used sparingly)

Only show one strand pattern per span (i.e. Do not show both raised and draped span alternatives for a given span).

A different girder spacing may need to be selected. It is often more economical to add an extra girder line than to maximize the number of strands and use debonding.

Prestressed box girders strands are to be straight, bonded, and located as shown in the Standard Details.

19.3.3.12.1 Raised Strand Patterns

Some of the standard strand patterns listed in the Standard Details show a raised strand pattern. Generally strands are placed so that the center of gravity of the strand pattern is as close as possible to the bottom of the girder. With a raised strand pattern, the center of gravity of the strand pattern is raised slightly and is a constant distance from the bottom of the girder for its entire length. Present practice is to show a standard raised arrangement as a preferred alternate to draping for short spans. For longer spans, debonding at the ends of the strands is an alternate (see 19.3.3.12.3). Use 0.6" strands for all raised patterns.

19.3.3.12.2 Draped Strand Patterns

Draping some of the strands is another available method to decrease stresses from prestress at the ends of the I-beam where the stress due to applied loads are minimum.

The typical strand profile for this technique is shown in Figure 19.3-1.

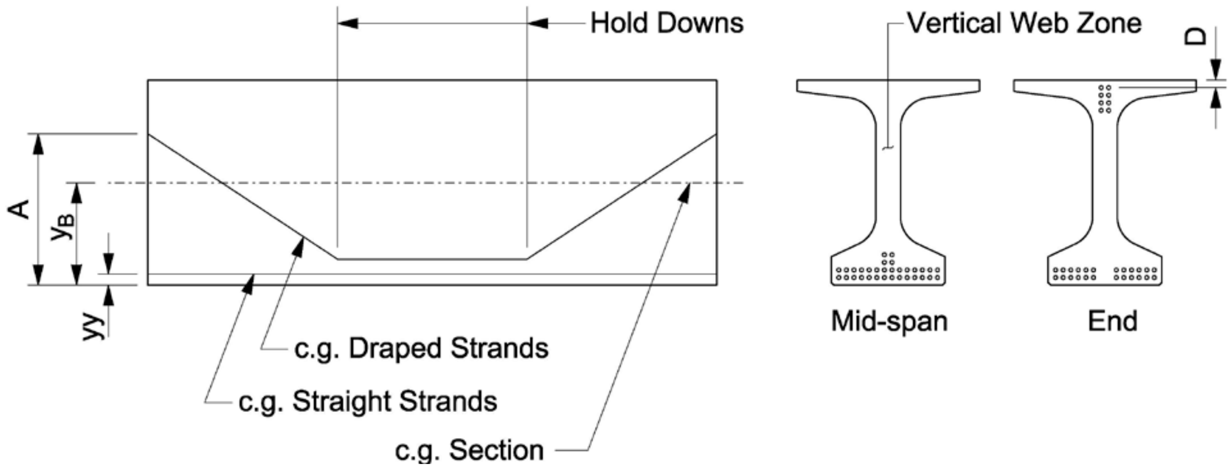


Figure 19.3-1
Typical Draped Strand Profile

Note that all the strands that lie within the "vertical web zone" of the mid-span arrangement are used in the draped group.

The engineer should show only one strand size for the draped pattern on the plans. Use only 0.5" strands for the draped pattern on 28" and 36" prestressed I-girders and 0.6" strands for all raised (straight) patterns for these shapes. Use 0.6" strands, only, for 36W", 45W", 54W", 72W" and 82W" prestressed I-girders. See Chapter 40 standards for 45", 54" and 70" prestressed I-girders.

Hold-down points for draped strands are located approximately between the 1/3 point and the 4/10 point from each end of the girder. The Standard Details, Prestressed Girder Details, show B values at the 1/4 point of the girder. On the plan sheets provide values for B_{min} and B_{max} as determined by the formulas shown on the Standards.

The maximum slope specified for draped strands is 12%. This limit is determined from the safe uplift load per strand of commercially available strand restraining devices used for hold-downs. The minimum distance, D, allowed from center of strands to top of flange is 2". For most designs, the maximum allowable slope of 12% will determine the location of the draped strands. Using a maximum slope will also have a positive effect on shear forces.

Initial girder stresses are checked at the end of the transfer length, which is located 60 strand diameters from the girder end. The transfer length is the embedment length required to develop f_{pe}, the effective prestressing steel stress (ksi) after losses. The prestressing steel stress varies linearly from 0.0 to f_{pe} along the transfer length.

The longer full development length of the strand is required to reach the larger prestressing steel stress at nominal resistance, f_{ps} (ksi). The strand stress is assumed to increase linearly from f_{pe} to f_{ps} over the distance between the transfer length and development length.

Per **LRFD [5.11.4.2]**, the development length is:

$$l_d \geq \kappa\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

d_b	=	Nominal strand diameter (in)
κ	=	1.0 for members with a depth less than or equal to 24”, and 1.6 for members with a depth of greater than 24”

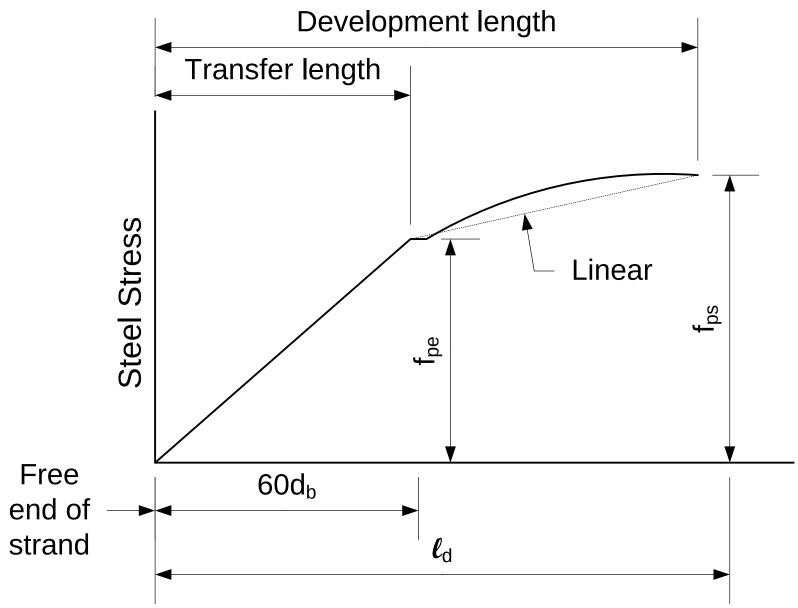


Figure 19.3-2
Transfer and Development Length

19.3.3.12.3 Partially Debonded Strand Patterns

The designer may use debonded strands if a raised or draped strand configuration fails to meet the allowable service stresses. The designer should exercise caution when using debonded strands as this may not result in the most economical design. Partially debonded strands are fabricated by wrapping sleeves around individual strands for a specified length from the ends of the girder, rendering the bond between the strand and the girder concrete ineffective for the wrapped, or shielded, length.

Bond breakers should only be applied to interior strands as girder cracking has occurred when they were applied to exterior strands. In computing bond breaker lengths, consideration is given to the theoretical stresses at the ends of the girder. These stresses are due entirely to prestress. As a result, the designer may compute a stress reduction based on certain strands having bond breakers. This reduction can be applied along the length of the debonded strands.

Partially debonded strands must adhere to the requirements listed in **LRFD [5.11.4.3]**. The list of requirements is as follows:

- The development length of partially debonded strands shall be calculated in accordance with **LRFD [5.11.4.2]** with $\kappa = 2.0$.
- The number of debonded strands shall not exceed 25% of the total number of strands.
- The number of debonded strands in any horizontal row shall not exceed 40% of the strands in that row.
- The length of debonding shall be such that all limit states are satisfied with consideration of the total developed resistance (transfer and development length) at any section being investigated.
- Not more than 40% of the debonded strands, or four strands, whichever is greater, shall have debonding terminated at any section.
- The strand pattern shall be symmetrical about the vertical axis of the girder. The consideration of symmetry shall include not only the strands being debonded but their debonded length as well, with the goal of keeping the center of gravity of the prestress force at the vertical centerline of the girder at any section. If the center of gravity of the prestress force deviates from the vertical centerline of the girder, the girder will twist, which is undesirable.
- Exterior strands in each horizontal row shall be fully bonded for crack control purposes.

19.3.3.13 Strength Limit State

The design factored positive moment is determined using the following equation:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

The Strength I limit state is applied to both simple and continuous span structures. See 17.2.4 for further information regarding loads and load combinations.

19.3.3.13.1 Factored Flexural Resistance

The nominal flexural resistance assuming rectangular behavior is given by **LRFD [5.7.3.2.3]** and **LRFD [5.7.3.2.2]**.

The section will act as a rectangular section as long as the depth of the equivalent stress block, a, is less than or equal to the depth of the compression flange (the structural deck thickness). Per **LRFD [5.7.3.2.2]**:

$$a = c\beta_1$$

Where:

c	=	Distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded (in)
β_1	=	Stress block factor **LRFD [5.7.2.2]**

By neglecting the area of mild compression and tension reinforcement, the equation presented in **LRFD [5.7.3.1.1]** for rectangular section behavior reduces to:

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f'_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Where:

A_{ps}	=	Area of prestressing steel (in^2)
f_{pu}	=	Specified tensile strength of prestressing steel (ksi)
f'_c	=	Compressive strength of the flange ($f'_{c(deck)}$ for rectangular section) (ksi)
b	=	Width of compression flange (in)
k	=	0.28 for low relaxation strand per **LRFD [C5.7.3.1.1]**
d_p	=	Distance from extreme compression fiber to the centroid of the prestressing tendons (in)
α_1	=	Stress block factor; equals 0.85 (for $f'_C \leq 10.0$ ksi) **LRFD [5.7.2.2]**

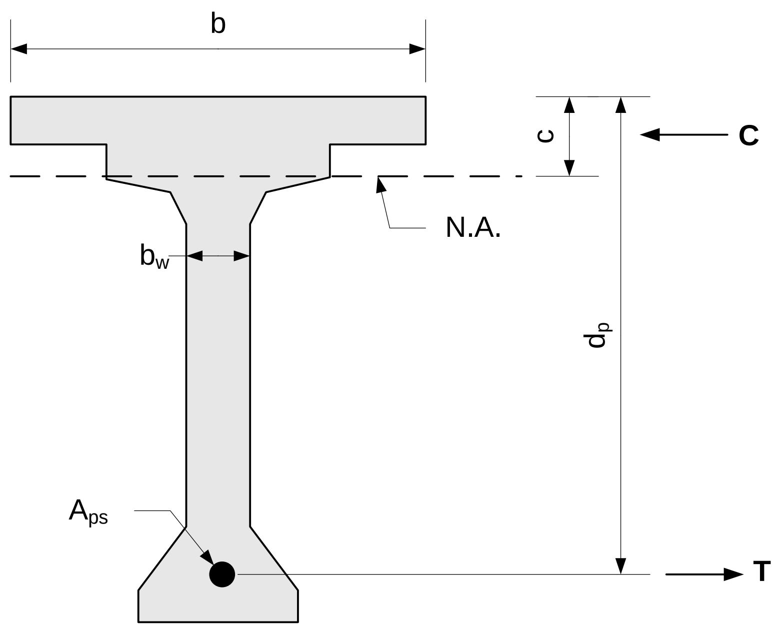


Figure 19.3-3
Depth to Neutral Axis, c

Verify that rectangular section behavior is allowed by checking that the depth of the equivalent stress block, a, is less than or equal to the structural deck thickness. If it is not, then T-section behavior provisions should be followed. If the T-section provisions are used, the compression block will be composed of two different materials with different compressive strengths. In this situation, **LRFD [C5.7.2.2]** recommends using β_1 and α_1 corresponding to the lower f'_c. The following equation for c shall be used for T-section behavior: **LRFD [5.7.3.1.1]**

$$c = \frac{A_{ps} f_{pu} - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w + k A_{ps} \frac{f_{pu}}{d_p}}$$

Where:

b_w	=	Width of web (in) – use the top flange width if the compression block does not extend below the haunch.
h_f	=	Depth of compression flange (in)

The factored flexural resistance presented in **LRFD [5.7.3.2.2]** is simplified by neglecting the area of mild compression and tension reinforcement. Furthermore, if rectangular section behavior is allowed, then $b_w = b$, where b_w is the web width as shown in Figure 19.3-3. The equation then reduces to:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

Where:

M_r	=	Factored flexural resistance (kip-in)
ϕ	=	Resistance factor
f_{ps}	=	Average stress in prestressing steel at nominal bending resistance (refer to **LRFD [5.7.3.1.1]**) (ksi)

If the T-section provisions must be used, the factored moment resistance equation is then:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + \alpha_1 \phi f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Where:

h_f	=	Depth of compression flange with width, b (in)

The engineer must then verify that M_r is greater than or equal to M_u.

<u>WisDOT exception to AASHTO:</u>

WisDOT standard prestressed I-girders and strand patterns are tension-controlled. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required when the standard girders and strand patterns are used, and $\phi = 1$.

19.3.3.13.2 Minimum Reinforcement

Per **LRFD [5.7.3.3.2]**, the minimum amount of prestressed reinforcement provided shall be adequate to develop a M_r at least equal to the lesser of M_{cr}, or $1.33M_u$.

M_{cr} is the cracking moment, and is given by:

$$M_{cr} = \gamma_3 \left[S_c (\gamma_1 f_r + \gamma_2 f_{cpe}) - 12M_{dnc} [(S_c/S_{nc}) - 1] \right]$$

Where:

S_c	=	Section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in^3)
f_r	=	Modulus of rupture (ksi)

f_{cpe} = Compressive stress in concrete due to effective prestress forces only (after losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

M_{dnc} = Total unfactored dead load moment acting on the basic beam (k-ft)

S_{nc} = Section modulus for the extreme fiber of the basic beam where tensile stress is caused by externally applied loads (in^3)

γ_1 = 1.6 flexural cracking variability factor

γ_2 = 1.1 prestress variability factor

γ_3 = 1.0 for prestressed concrete structures

Per **LRFD [5.4.2.6]**, the modulus of rupture for normal weight concrete is given by:

$f_r = 0.24\lambda\sqrt{f'_c}$; where λ = conc. density modification factor **LRFD [5.4.2.8]**, and has a value of 1.0 for normal weight conc.

19.3.3.14 Non-prestressed Reinforcement

Non-prestressed reinforcement consists of bar steel reinforcement used in the conventional manner. It is placed longitudinally along the top of the member to carry any tension which may develop after transfer of prestress. The designer should completely detail all rebar layouts including stirrups.

The amount of reinforcement is that which is sufficient to resist the total tension force in the concrete based on the assumption of an uncracked section.

For draped designs, the control is at the hold-down point of the girder. At the hold-down point, the initial prestress is acting together with the girder dead load stress. This is where tension due to prestress is still maximum and compression due to girder dead load is decreasing.

For non-draped designs, the control is at the end of the member where prestress tension exists but dead load stress does not.

Note that a minimum amount of reinforcement is specified in the Standards. This is intended to help prevent serious damage due to unforeseeable causes like improper handling or storing.

19.3.3.15 Horizontal Shear Reinforcement

The horizontal shear reinforcement resists the Strength I limit state horizontal shear that develops at the interface of the slab and girder in a composite section. The dead load used to calculate the horizontal shear should only consider the DC and DW dead loads that act on the composite section. See 17.2.4 for further information regarding the treatment of dead loads and load combinations.

$$V_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

$$V_{ni} \geq V_{ui} / \phi$$

Where:

V_u	=	Maximum strength limit state vertical shear (kips)
V_{ui}	=	Strength limit state horizontal shear at the girder/slab interface (kips)
V_{ni}	=	Nominal interface shear resistance (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2.1]**

The shear stress at the interface between the slab and the girder is given by:

$$v_{ui} = \frac{V_u}{b_{vi} d_v}$$

Where:

v_{ui}	=	Factored shear stress at the slab/girder interface (ksi)
b_{vi}	=	Interface width to be considered in shear transfer (in)
d_v	=	Distance between the centroid of the girder tension steel and the mid-thickness of the slab (in)

The factored horizontal interface shear shall then be determined as:

$$V_{ui} = 12 v_{ui} b_{vi}$$

The nominal interface shear resistance shall be taken as:

$$V_{ni} = cA_{cv} + \mu\left[A_{vf} f_y + P_c\right]$$

Where:

A_{cv}	=	Concrete area considered to be engaged in interface shear transfer. This value shall be set equal to $12b_{vi}$ (ksi)
c	=	Cohesion factor specified in **LRFD [5.8.4.3]**. This value shall be taken as 0.28 ksi for WisDOT standard girders with a cast-in-place deck

μ	=	Friction factor specified in **LRFD [5.8.4.3]**. This value shall be taken as 1.0 for WisDOT standard girders with a cast-in-place deck (dim.)
A_{vf}	=	Area of interface shear reinforcement crossing the shear plan within the area A_{cv} (in^2)
f_y	=	Yield stress of shear interface reinforcement not to exceed 60 (ksi)
P_c	=	Permanent net compressive force normal to the shear plane (kips)

P_c shall include the weight of the deck, haunch, parapets, and future wearing surface. A conservative assumption that may be considered is to set $P_c = 0.0$.

The nominal interface shear resistance, V_{ni}, shall not exceed the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv} \quad \text{or} \quad V_{ni} \leq K_2 A_{cv}$$

Where:

K_1	=	Fraction of concrete strength available to resist interface shear as specified in **LRFD [5.8.4.3]**. This value shall be taken as 0.3 for WisDOT standard girders with a cast-in-place deck (dim.)
K_2	=	Limiting interface shear resistance as specified in **LRFD [5.8.4.3]**. This value shall be taken as 1.8 ksi for WisDOT standard girders with a cast-in-place deck

WisDOT policy item:

The stirrups that extend into the deck slab presented on the Standards are considered adequate to satisfy the minimum reinforcement requirements of **LRFD [5.8.4.4]**

19.3.3.16 Web Shear Reinforcement

Web shear reinforcement consists of placing conventional reinforcement perpendicular to the axis of the girder.

WisDOT policy item:

Web shear reinforcement shall be designed by **LRFD [5.8.3.4.3]** (Simplified Procedure) using the Strength I limit state for WisDOT standard girders.

WisDOT prefers girders with spacing symmetrical about the midspan in order to simplify design and fabrication. The designer is encouraged to simplify the stirrup arrangement as much as possible. For vertical stirrups, the required area of web shear reinforcement is given by the following equation:

$$A_v \geq \frac{(V_n - V_c)s}{f_y d_v \cot\theta} \quad \text{(or } 0.0316\lambda\sqrt{f'_c}\frac{b_v s}{f_y} \text{ minimum , } \textbf{LRFD [5.8.2.5]}\text{)}$$

Where:

A_v	=	Area of transverse reinforcement within distance, s (in^2)
V_n	=	Nominal shear resistance (kips)
V_c	=	Nominal shear resistance provided by tensile stress in the concrete (kips)
s	=	Spacing of transverse reinforcement (in)
f_y	=	Specified minimum yield strength of transverse reinforcement (ksi)
d_v	=	Effective shear depth as determined in **LRFD [5.8.2.9]** (in)
b_v	=	Minimum web width within depth, d_v
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

$\cot\theta$ shall be taken as follows:

- When $V_{ci} < V_{cw}$, $\cot\theta = 1.0$
- When $V_{ci} > V_{cw}$, $\cot\theta = 1.0 + 3\left(\frac{f_{pc}}{\sqrt{f'_c}}\right) \leq 1.8$

$$V_u = 1.25DC + 1.5DW + 1.75(LL + IM)$$
$$V_n = V_u / \phi$$

Where:

V_u	=	Strength I Limit State shear force (kips)
ϕ	=	0.90 per **LRFD [5.5.4.2.1]**

See 17.2 for further information regarding load combinations.

Per **LRFD [5.8.3.4.3]**, determine V_c as the minimum of either V_{ci} or V_{cw} given by:

$$V_{cw} = (0.06\lambda\sqrt{f'_c} + 0.30f_{pc})b_v d_v + V_p$$

$$V_{ci} = 0.02\lambda\sqrt{f'_c}b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\lambda\sqrt{f'_c}b_v d$$

Where:

f_{pc}	=	Compressive stress in concrete, after all prestress losses, at centroid of cross section resisting externally applied loads or at the web-flange junction when the centroid lies within the flange. (ksi) In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the web-flange junction, due to both prestress and moments resisted by the member acting alone.
V_d	=	Shear force at section due to unfactored dead loads (kips)
V_i	=	Factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kips)
M_{cre}	=	Moment causing flexural cracking at the section due to externally applied loads (k-in)
M_{max}	=	Maximum factored moment at section due to externally applied loads (k-in)
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

$$V_i = V_u - V_d$$

$$M_{cre} = S_c\left(f_r + f_{cpe} - \frac{12M_{dnc}}{S_{nc}}\right)$$

$$M_{max} = M_u - M_{dnc}$$

Where:

S_c	=	Section modulus for the extreme tensile fiber of the composite section where the stress is caused by externally applied loads (in^3)
S_{nc}	=	Section modulus for the extreme tensile fiber of the non-composite section where the stress is caused by externally applied loads (in^3)
f_{cpe}	=	Compressive stress in concrete due to effective prestress forces only, after all prestress losses, at the extreme tensile fiber of the section where the stress is caused by externally applied loads (ksi)
M_{dnc}	=	Total unfactored dead load moment acting on the non-composite section (k-ft)
f_r	=	Modulus of rupture of concrete. Shall be $= 0.20\lambda\sqrt{f'_c}$ (ksi) **LRFD [5.4.2.6]**
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

For a composite section, V_{ci} corresponds to shear at locations of accompanying flexural stress. V_{cw} corresponds to shear at simple supports and points of contraflexure. The critical computation for V_{cw} is at the centroid for composite girders.

Set the vertical component of the draped strands, V_p, equal to 0.0 when calculating V_n, as per **LRFD [5.8.3.3]**. This vertical component helps to reduce the shear on the concrete section. The actual value of V_p should be used when calculating V_{cw}. However, the designer may make the conservative assumption to neglect V_p for all shear resistance calculations.

WisDOT policy item:

Based on past performance, for prestressed I-girders the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.8.2.7]** will be reduced to 18 inches.

When determining shear reinforcement, spacing requirements as determined by analysis at 1/10th points, for example, should be carried-out to the next 1/10th point. As an illustration, spacing requirements for the 1/10th point should be carried out to very close to the 2/10th point, as the engineer, without a more refined analysis, does not know what the spacing requirements would be at the 0.19 point. For the relatively small price of stirrups, don't shortchange the shear capacity of the prestressed girder.

The web reinforcement spacing shall not exceed the maximum permitted spacing determined as:

- If $\upsilon_u < 0.125f'_c$, then $s_{max} = 0.8d_v \leq 18"$
- If $\upsilon_u \geq 0.125f'_c$, then $s_{max} = 0.4d_v \leq 12"$

Where:

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ per **LRFD [5.8.2.9]**.

The nominal shear resistance, $V_c + V_s$, is limited by the following:

$$V_c + \frac{A_v f_y d_v \cot\theta}{s} \leq 0.25f'_c\, b_v d_v$$

Reinforcement in the form of vertical stirrups is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at transfer at a unit stress of 20 ksi and are placed within h/4 of the girder end, where h is the total girder depth. For a distance of 1.5d from the ends of the beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, shall be a #3 bar or greater and shall be spaced at less than or equal to 6". Note that the reinforcement shown on the Standard Details sheets satisfies these requirements.

Welded wire fabric may be used for the vertical reinforcement. It must be deformed wire with a minimum size of D18.

Per **LRFD [5.8.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi} - 0.5V_s \right) \cot\theta$$

In the above equation, $\cot\theta$ is as defined in the V_c discussion above, and V_s is the shear reinforcement resistance at the section considered. Any lack of full reinforcement development shall be accounted for. Note that the reinforcement shown on the Standard Detail sheets satisfies these requirements.

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.7.3]**:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

LRFD [5.5.4.2] allows a ϕ factor equal to 0.9 for tension-controlled reinforced concrete sections such as the bridge deck.

The continuity reinforcement consists of mild steel reinforcement in the deck in the negative moment region over the pier. Consider both the non-composite and the superimposed dead loads and live loads for the Strength I design of the continuity reinforcement in the deck.

Moment resistance is developed in the same manner as shown in 19.3.3.13.1 for positive moments, except that the bottom girder flange is in compression and the deck is in tension. The moment resistance is formed by the couple resulting from the compression force in the bottom flange and the tension force from the longitudinal deck steel. Consider A_s to consist of the longitudinal deck steel present in the deck slab effective flange width as determined in 19.3.3.8. The distance, d_p, is taken from the bottom of the girder flange to the center of the longitudinal deck steel.

WisDOT exception to AASHTO:

Composite sections formed by WisDOT standard prestressed I-girders shall be considered to be tension-controlled for the design of the continuity reinforcement. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required, and $\phi = 0.9$.

WisDOT policy item:

New bridge designs shall consider only the top mat of longitudinal deck steel when computing the continuity reinforcement capacity.

WisDOT policy item:

The continuity reinforcement shall be based on the greater of either the interior girder design or exterior girder and detailed as typical reinforcement for the entire width of the bridge deck. However, do not design the continuity steel based on the exterior girder design beneath a raised sidewalk. The continuity steel beneath a raised sidewalk should not be used for rating.

Based on the location of the neutral axis, the bottom flange compressive force may behave as either a rectangle or a T-section. On WisDOT standard prestressed I-girders, if the depth of the compression block, a, falls within the varying width of the bottom flange, the compression block acts as an idealized T-section. In this case, the width, b, shall be taken as the bottom flange width, and the width, b_w, shall be taken as the bottom flange width at the depth “a”. During T-section behavior, the depth, h_f, shall be taken as the depth of the bottom flange of full width, b. See Figure 19.3-4 for details. Ensure that the deck steel is adequate to satisfy $M_r \geq M_u$.

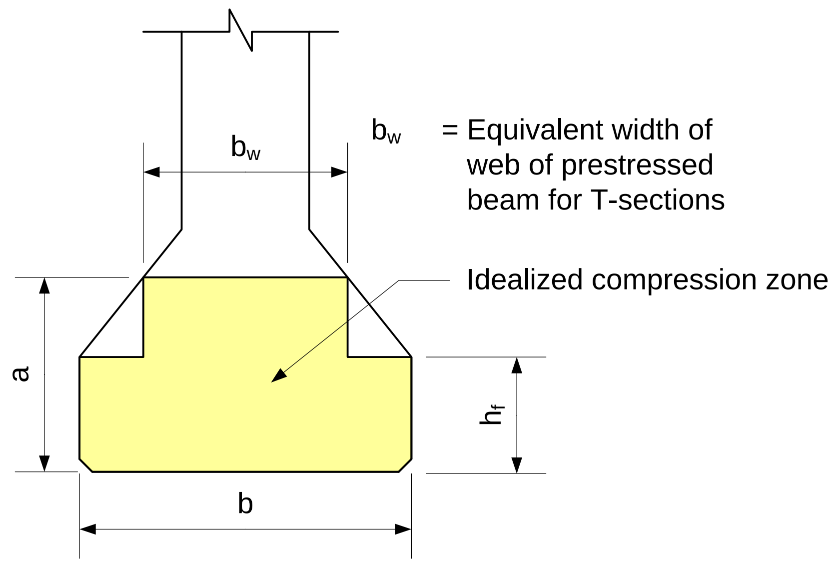


Figure 19.3-4
T-Section Compression Flange Behavior

The continuity reinforcement should also be checked to ensure that it meets the crack control provisions of **LRFD [5.7.3.4]**. This check shall be performed assuming severe exposure conditions. Only the superimposed loads shall be considered for the Service and Fatigue requirements.

The concrete between the abutting girder ends is usually of a much lesser strength than that of the girders. However, tests[1] have shown that, due to lateral confinement of the diaphragm concrete, the girder itself fails in ultimate negative compression rather than failure in the material between its ends. Therefore the ultimate compressive stress, f'_c, of the girder concrete is used in place of that of the diaphragm concrete.

This assumption has only a slight effect on the computed amount of reinforcement, but it has a significant effect on keeping the compression force within the bottom flange.

The continuity reinforcement shall conform to the Fatigue provisions of **LRFD [5.5.3]**.

The transverse spacing of the continuity reinforcement is usually taken as the whole or fractional spacing of the D bars as given in 17.5.3.2. Grade 60 bar steel is used for continuity reinforcement. Required development lengths for deformed bars are given in Chapter 9 – Materials.

WisDOT exception to AASHTO:

The continuity reinforcement is not required to be anchored in regions of the slab that are in compression at the strength limit state as stated in **LRFD [5.14.1.4.8]**. The following locations shall be used as the cut off points for the continuity reinforcement:

1. When ½ the bars satisfy the Strength I moment envelope (considering both the non-composite and composite loads) as well as the Service and Fatigue moment envelopes (considering only the composite moments), terminate ½ of the bars. Extend these bars past this cutoff point a distance not less than the girder depth or 1/16 the clear span for embedment length requirements.

2. Terminate the remaining one-half of the bars an embedment length beyond the point of inflection. The inflection point shall be located by placing a 1 klf load on the composite structure. This cut-off point shall be at least 1/20 of the span length or 4' from point 1, whichever is greater.

Certain secondary features result when spans are made continuous. That is, positive moments develop over piers due to creep[5], shrinkage and the effects of live load and dynamic load allowance in remote spans. The latter only exists for bridges with three or more spans.

These positive moments are somewhat counteracted by negative moments resulting from differential shrinkage[4] between the cast-in-place deck and precast girders along with negative moments due to superimposed dead loads. However, recent field observations cited in **LRFD [C5.14.1.4.2]** suggest that these moments are less than predicted by analysis. Therefore, negative moments caused by differential shrinkage should be ignored in design.

WisDOT exception to AASHTO:

WisDOT requires the use of a negative moment connection only. The details for a positive moment connection per **LRFD [5.14.1.4]** are not compatible with the Standard Details and should not be provided.

19.3.3.18 Camber and Deflection

The prestress camber and dead load deflection are used to establish the vertical position of the deck forms with respect to the girder. The theory presented in the following sections apply to a narrow set of circumstances. The designer is responsible for ensuring that the theoretical camber accounts for the loads applied to the girder. For example, if the diaphragms of a prestressed I-girder are configured so there is one at each of the third points instead of one at midspan, the term in the equation for $\Delta_{nc(DL)}$ related to the diaphragms in 19.3.3.18.2 would need to be modified to account for two point loads applied at the third points instead of one point load applied at midspan.

Deflection effects due to individual loads may be calculated separately and superimposed, as shown in this section. The *PCI Design Handbook* provides design aids to assist the designer in the evaluation of camber and deflection, including cambers for prestress forces and loads, and beam design equations and diagrams.

Figure 19.3-5 illustrates a typical prestressed I-girder with a draped strand profile.

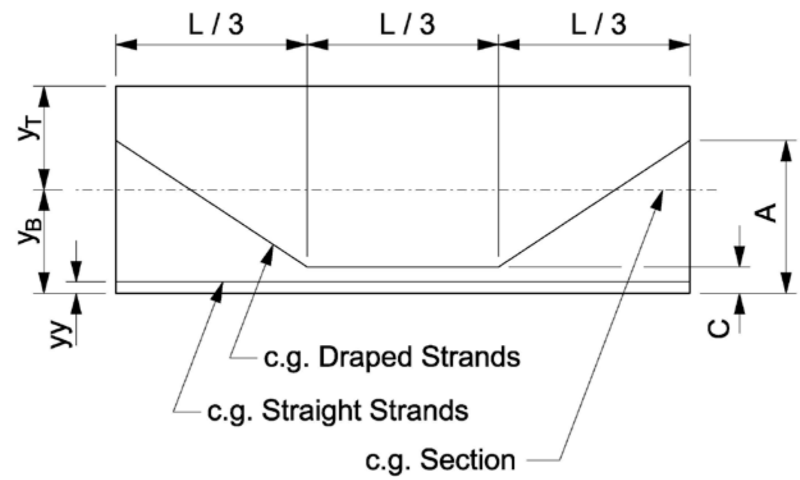


Figure 19.3-5
Typical Draped Strand Profile

19.3.3.18.1 Prestress Camber

The prestressing strands produce moments in the girder as a result of their eccentricity and draped pattern. These moments induce a camber in the girder. The values of the camber are calculated as follows:

Eccentric straight strands induce a constant moment of:

$$M_1 = \frac{1}{12}\left(P_i^s (y_B - yy)\right)$$

Where:

M_1	=	Moment due to initial prestress force in the straight strands minus the elastic shortening loss (k-ft)
P_i^s	=	Initial prestress force in the straight strands minus the elastic shortening loss (kips)
y_B	=	Distance from center of gravity of beam to bottom of beam (in)

yy = Distance from center of gravity of straight strands to bottom of beam (in)

This moment produces an upward deflection at midspan which is given by:

$$\Delta_s = \frac{M_1 L^2}{8 E_i I_b} \quad \text{(with all units in inches and kips)}$$

For moments expressed in kip-feet and span lengths expressed in feet, this equation becomes the following:

$$\Delta_s = \frac{M_1 L^2}{8 E_i I_b}\left(\frac{12}{1}\right)\left(\frac{12^2}{1}\right) = \frac{M_1 L^2}{8 E_i I_b}\left(\frac{1728}{1}\right)$$

$$\Delta_s = \frac{216 M_1 L^2}{E_i I_b} \quad \text{(with units as shown below)}$$

Where:

Δ_s	=	Deflection due to force in the straight strands minus elastic shortening loss (in)
L	=	Span length between centerlines of bearing (ft)
E_i	=	Modulus of elasticity at the time of release (see 19.3.3.8) (ksi)
I_b	=	Moment of inertia of basic beam (in^4)

The draped strands induce the following moments at the ends and within the span:

$M_2 = \frac{1}{12}\left(P_i^D (A - C)\right)$, which produces upward deflection, and

$M_3 = \frac{1}{12}\left(P_i^D (A - y_B)\right)$, which produces downward deflection when A is greater than y_B

Where:

M_2, M_3	=	Components of moment due to initial prestress force in the draped strands minus the elastic shortening loss (k-ft)
P_i^D	=	Initial prestress force in the draped strands minus the elastic shortening loss (kips)
A	=	Distance from bottom of beam to center of gravity of draped strands at centerline of bearing (in)
C	=	Distance from bottom of beam to center of gravity of draped strands between hold-down points (in)

These moments produce a net upward deflection at midspan, which is given by:

$$\Delta_D = \frac{216L^2}{E_i I_b}\left(\frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_D = Deflection due to force in the draped strands minus elastic shortening loss (in)

The combined upward deflection due to prestress is:

$$\Delta_{PS} = \Delta_s + \Delta_D = \frac{216L^2}{E_i I_b}\left(M_1 + \frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_{PS} = Deflection due to straight and draped strands (in)

The downward deflection due to beam self-weight at release is:

$$\Delta_{o(DL)} = \frac{5W_bL^4}{384E_iI_b} \quad \text{(with all units in inches and kips)}$$

Using unit weights in kip per foot, span lengths in feet, E in ksi and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_bL^4}{384E_iI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) = \frac{5W_bL^4}{384E_iI_b}\left(\frac{20736}{12}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{E_iI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{o(DL)}$ = Deflection due to beam self-weight at release (in)

W_b = Beam weight per unit length (k/ft)

Therefore, the anticipated prestress camber at release is given by:

$$\Delta_i = \Delta_{PS} - \Delta_{o(DL)}$$

Where:

Δ_i = Prestress camber at release (in)

Camber, however, continues to grow after the initial strand release. For determining substructure beam seats, average concrete haunch values (used for both DL and quantity calculations) and the required projection of the vertical reinforcement from the tops of the prestressed girders, **a camber multiplier of 1.4 shall be used**. This value is multiplied by the theoretical camber at release value.

19.3.3.18.2 Dead Load Deflection

The downward deflection of a prestressed I-girder due to the dead load of the deck and a midspan diaphragm is:

$$\Delta_{nc\,(DL)} = \frac{5W_{deck}L^4}{384EI_b} + \frac{P_{dia}L^3}{48EI_b} \quad \text{(with all units in inches and kips)}$$

Using span lengths in units of feet, unit weights in kips per foot, E in ksi, and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{12^3}{1}\right) = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{20736}{12}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{1728}{1}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{EI_b} + \frac{36P_{dia}L^3}{EI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{nc\,(DL)}$ = Deflection due to non-composite dead load (deck and midspan diaphragm) (in)

W_{deck} = Deck weight per unit length (k/ft)

P_{dia} = Midspan diaphragm weight (kips)

E = Girder modulus of elasticity at final condition (see 19.3.3.8) (ksi)

A similar calculation is done for parapet and sidewalk loads on the composite section. Provisions for deflections due to future wearing surface shall not be included.

For girder structures with raised sidewalks, loads shall be distributed as specified in Chapter 17, and separate deflection calculations shall be performed for the interior and exterior girders.

19.3.3.18.3 Residual Camber

Residual camber is the camber that remains after the prestress camber has been reduced by the composite and non-composite dead load deflection. Residual camber is computed as follows:

$$RC = \Delta_i - \Delta_{nc(DL)} - \Delta_{c(DL)}$$

19.3.4 Prestressed I-Girder Deck Forming

Deck forming requires computing the relationship between the top of girder and bottom of deck necessary to achieve the desired vertical roadway alignment. Current practice for design is to use a minimum haunch of 2" at the edge of the girder flange. This haunch value is also used for calculating composite section properties. This will facilitate current deck forming practices which use 1/2" removable hangers and 3/4" plywood, and it will allow for variations in prestress camber. Also, future deck removal will be less likely to damage the top girder flanges. An average haunch height of 3 inches minimum can be used for determining haunch weight for preliminary design. It should be noted that the actual haunch values should be compared with the estimated values during final design. If there are significant differences in these values, the design should be revised. The actual average haunch height should be used to calculate the concrete quantity reported on the plans as well as the value reported on the prestressed girder details sheet. The actual haunch values at the girder ends shall be used for determining beam seat elevations.

For designs involving vertical curves, Figure 19.3-6 shows two different cases.

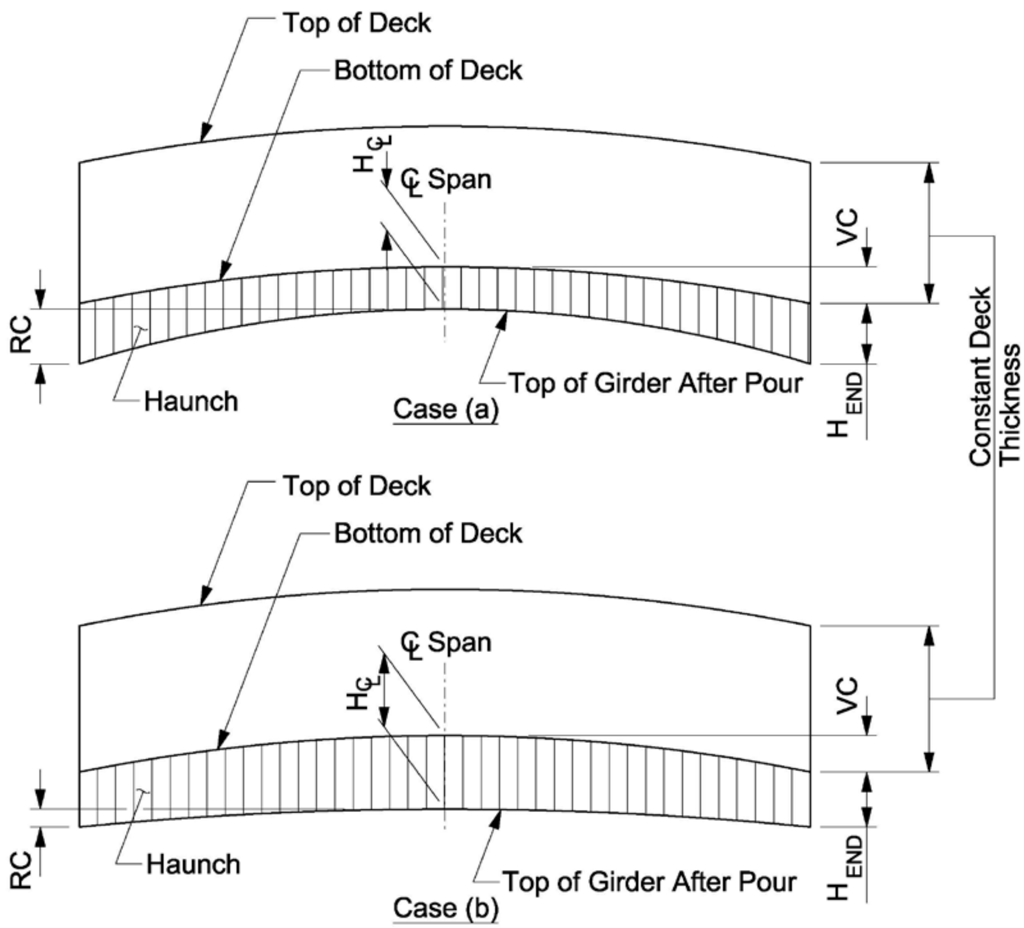


Figure 19.3-6
Relationship between Top of Girder and Bottom of Deck

In Case (a), VC is less than the computed residual camber, RC, and the minimum haunch occurs at midspan. In Case (b), VC is greater than RC and the minimum haunch occurs at the girder ends.

Deck forms are set to accommodate the difference between the bottom of the deck and the top of the girder under all dead loads placed at the time of construction, including the wet deck concrete and superimposed parapet and sidewalk loads. The deflection of superimposed future wearing surface and live loads are not included.

19.3.4.1 Equal-Span Continuous Structures

For equal-span continuous structures having all spans on the same vertical alignment, the deck forming is the same for each span. This is due to the constant change of slope of the vertical curve or tangent and the same RC per span.

The following equation is derived from Figure 19.3-6:

$$+H_{END} = RC - VC + (+H_{CL})$$

Where:

H_{END}	=	See Figure 19.3-6 (in)
RC	=	Residual camber, positive for upward (in)
VC	=	Difference in vertical curve, positive for crest vertical curves and negative for sag vertical curves (in)
H_{CL}	=	See Figure 19.3-6 (in)

19.3.4.2 Unequal Spans or Curve Combined With Tangent

For unequal spans or when some spans are on a vertical curve and others are on a tangent, a different approach is required. Generally the longer span or the one off the curve dictates the haunch required at the common support. Therefore, it is necessary to pivot the girder about its midspan in order to achieve an equal condition at the common support. This is done mathematically by adding together the equation for each end (abutment and pier), as follows:

$$(+H_{LT}) + (+H_{RT}) = 2[RC - VC + (+H_{CL})]$$

Where:

H_{LT}	=	H_{END} at left (in)
H_{RT}	=	H_{END} at right (in)

With the condition at one end known due to the adjacent span, the condition at the other end is computed.

19.3.5 Construction Joints

The transverse construction joints should be located in the deck midway between the cut-off points of the continuity reinforcement or at the 0.75 point of the span, whichever is closest to the pier. The construction joint should be located at least 1' from the cut-off points.

This criteria keeps stresses in the slab reinforcement due to slab dead load at a minimum and makes deflections from slab dead load closer to the theoretical value.

19.3.6 Strand Types

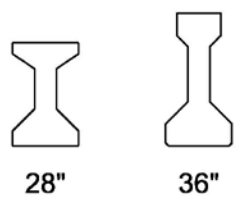
Low relaxation strands (0.5” and 0.6” in diameter) are currently used in prestressed I-girder and prestressed box girder designs and are shown on the plans. Strand patterns and initial prestressing forces are given on the plans, and deflection data is also shown.

19.3.7 Construction Dimensional Tolerances

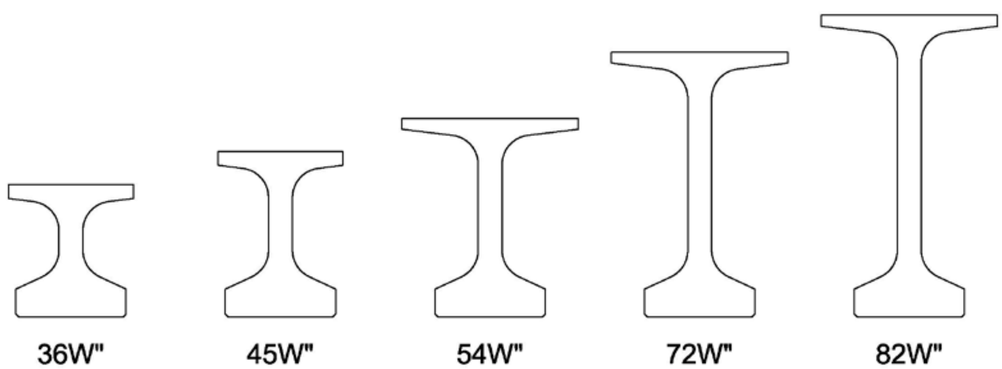
Refer to the *AASHTO LRFD Bridge Construction Specifications* for the required dimensional tolerances.

19.3.8 Prestressed I-Girder Sections

WisDOT BOS employs two prestress I-girder section families. One I section family follows the AASHTO standard section, while the other section family follows a wide flange bulb-tee, see Figure 19.3-7. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. For these sections, the cost of draping far exceeds savings in strands. See the Standard Details for the prestressed I-girder sections' draped and undraped strand patterns. Note, for the 28" prestressed I-girder section the 16 and 18 strand patterns require bond breakers.



WisDOT Standard Girder Shapes



WisDOT Wide Flange Girder Shapes

Figure 19.3-7
Prestressed I-Girder Family Details

Table 19.3-1 and Table 19.3-2 provide span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder sections. Girder spacings are based on using low relaxation strands at $0.75f_{pu}$, concrete haunch thicknesses, slab thicknesses from Chapter 17 – Superstructures - General and a future wearing surface. For these tables, a line load of 0.300 klf is applied to the girder to account for superimposed dead loads.

Several girder shapes have been retired from standard use on new structures. These include the following sizes; 45-inch, 54-inch and 70-inch. These girder shapes are used for girder replacements, widening and for curved new structures where the wide flange sections are not practical. See Chapter 40 – Bridge Rehabilitation for additional information on these girder shapes.

Due to the wide flanges on the 54W, 72W and 82W and the variability of residual camber, haunch heights frequently exceed 2”. An average haunch of 4” was used for all wide flange girders in the following tables. **Do not push the span limits/girder spacing during preliminary design.** See Table 19.3-2 for guidance regarding use of excessively long prestressed I-girders.

Tables are based on:

- Interior prestressed I-girders, 0.5” or 0.6” dia. strands (in accordance with the Standard Details).
- f'_c girder = 8,000 psi
- f'_c slab = 4,000 psi
- Haunch height (dead load) = 2 ½” for 28” and 36” girders

  = 4” for 45W”,54W”,72W” and 82W” girders
- Haunch height (section properties) = 2”
- Required f'_c girder at initial prestress < 6,800 psi

28" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	54	60
6’-6”	54	58
7’-0”	52	56
7’-6”	50	54
8’-0”	50	54
8’-6”	48	52
9’-0”	48	50
9’-6”	46	50
10’-0”	44	48
10’-6”	44	48
11’-0”	42	46
11’-6”	42	46
12’-0”	42	44

36" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	72	78
6’-6”	70	76
7’-0”	70	74
7’-6”	68	72
8’-0”	66	70
8’-6”	64	68
9’-0”	62	68
9’-6”	60	64
10’-0”	60	64
10’-6”	58	62
11’-0”	50	60
11’-6”	50	60
12’-0”	48	58

36W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	94	101
6’-6”	92	99
7’-0”	88	97
7’-6”	87	95
8’-0”	85	93
8’-6”	83	91
9’-0”	82	88
9’-6”	80	86
10’-0”	77	84
10’-6”	76	83
11’-0”	74	81
11’-6”	73	78
12’-0”	71	76

45W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	110	120
6’-6”	109	117
7’-0”	107	115
7’-6”	103	113
8’-0”	101	111
8’-6”	99	108
9’-0”	97	104
9’-6”	95	102
10’-0”	92	100
10’-6”	90	98
11’-0”	88	96
11’-6”	87	93
12’-0”	85	91

Table 19.3-1
Maximum Span Length vs. Girder Spacing

54W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	125	134
6’-6”	123	132
7’-0”	120	129
7’-6”	118	127
8’-0”	116	125
8’-6”	114	122
9’-0”	112	120
9’-6”	110	117
10’-0”	108	115
10’-6”	106	114
11’-0”	102	111
11’-6”	101	109
12’-0”	99	107

72W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	153*	164*⊗
6’-6”	150	161*⊗
7’-0”	148	158*
7’-6”	145	156*
8’-0”	143	153*
8’-6”	140	150
9’-0”	138	148
9’-6”	135	144
10’-0”	133	142
10’-6”	131	140
11’-0”	129	137
11’-6”	127	135
12’-0”	124	132

82W” Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	166*⊗	177*⊗
6’-6”	163*⊗	174*⊗
7’-0”	161*⊗	172*⊗
7’-6”	158*	169*⊗
8’-0”	156*	166*⊗
8’-6”	152	163*⊗
9’-0”	150	160*⊗
9’-6”	147	157*
10’-0”	145	154*
10’-6”	143	152
11’-0”	140	149
11’-6”	136	147
12’-0”	133	144

Table 19.3-2

Maximum Span Length vs. Girder Spacing

* For lateral stability during lifting these girder lengths may require pick-up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange, if required, and check the concrete strength near the

lift location based on f'_{ci}. A note should be placed on the girder details sheet to reflect that the girder was analyzed for a potential lift at the 1/10 point.

⊗ Due to difficulty manufacturing, transporting and erecting excessively long prestressed girders, consideration should be given to utilizing an extra pier to minimize use of such girders. Approval from the Bureau of Structures is required to utilize any girder over 158 ft. long. (Currently, there is still a moratorium on the use of all 82W"). Steel girders may be considered if the number of piers can be reduced enough to offset the higher costs associated with a steel superstructure.

19.3.8.1 Prestressed I-Girder Standard Strand Patterns

The standard strand patterns presented in the Standard Details were developed to eliminate some of the trial and error involved in the strand pattern selection process. These standard strand patterns should be used whenever possible, with a straight strand arrangement preferred over a draped strand arrangement. The designer is responsible for ensuring that the selected strand pattern meets all LRFD requirements.

Section 19.3.3 discusses the key parts of the design procedure, and how to effectively use the standard strand patterns along with Table 19.3-1 and Table 19.3-2.

The amount of drape allowed is controlled by the girder size and the 2" clearance from center of strand to top of girder. See the appropriate Standard Girder Details for guidance on draping.

19.3.9 Prestressed Box Girders Post-Tensioned Transversely

These sections may be used for skews up to 30° with the transverse post-tensioning ducts placed along the skew. Skews over 30° are not recommended, but if absolutely required the transverse post-tensioning ducts should be placed perpendicular to the prestressed sections. Also for skews over 30° a more refined method of analysis should be used such as an equivalent plate analysis or a finite element analysis.

Details for transverse post-tensioning are shown in the Standard Details. Each post-tensioning duct contains three ½" diameter strands which produce a total post-tensioning force per duct of 86.7 kips.

Prestressed box girders are subject to high chloride ion exposure because of longitudinal cracking that sometimes occurs between the boxes or from drainage on the fascia girders when an open steel railing system is used. To reduce permeability the concrete mix is required to contain fly ash as stated in 503.2.2 of the Standard Specifications.

When these sections are in contact with water for 5-year flood events or less, the sections must be cast solid for long term durability. When these sections are in contact with water for the 100-year flood event or less, any voids in the section must be cast with a non-water-absorbing material.

Table 19.3-3 provides approximate span limitations for prestressed box girder sections. It also gives the section properties associated with these members. Criteria for developing these tables are shown below Table 19.3-3.

19.3.9.1 Available Prestressed Box Girder Sections and Maximum Span Lengths

Precasters have forms available to make six prestressed girder box sections ranging in depth from 12" to 42". Each section can be made in widths of 36" and 48", but 48" is more efficient and is the preferred width. Typical box section information is shown in the Standard Details.

Table 19.3-3 shows available section depths, section properties, and maximum span length. All sections have voids except the 12" deep section.

	Section No.	Section Depth (inches)	Section Area, A, (in^2)	Moment of Inertia, I, (in^4)	Section Modulus, (in^3) S_{Top}	Section Modulus, (in^3) S_{Bottom}	Torsional Inertia, J, (in^4)	Max. Span (ft)
3'-0" Section Width	1	12	422	5,101	848	852	15,955	24
	2	17	452	14,047	1,648	1,657	23,797	40
	3	21	492	25,240	2,398	2,410	39,928	49
	4	27	565	50,141	3,706	3,722	68,925	58
	5	33	625	85,010	5,142	5,162	102,251	64
	6	42	715	158,749	7,546	7,573	158,033	77
4'-0" Section Width	1	12	566	6,829	1,136	1,140	22,600	25
	2	17	584	18,744	2,201	2,210	38,427	39
	3	21	624	33,501	3,184	3,197	65,395	49
	4	27	697	65,728	4,860	4,877	114,924	59
	5	33	757	110,299	6,674	6,696	173,031	68
	6	42	847	203,046	9,655	9,683	272,267	80

Table 19.3-3
Prestressed Box Girder Section Properties and Maximum Span Length

Table based on:

- HL93 loading and AASHTO LRFD Bridge Design Specifications
- Simple span
- f'_c = 5 ksi and f'_{ci} = 4.25 ksi
- 0.5" dia. or 0.6" dia., low relaxation prestressing strands at $0.75f'_s$
- f'_s = 270.0 ksi
- 6" min. concrete deck (which doesn't contribute to stiffness of section)
- Single slope parapet 42SS weight distributed evenly to all girder sections
- 30° skew used to compute diaphragm weight

- 2 ¾" of grout between sections
- Post-tensioning diaphragms located as stated in the Standard Details
- 30'-0" minimum clear bridge width (eleven 3'-0" sections, eight 4'-0" sections)

19.3.9.2 Decks and Overlays

There are three types of systems.

1. Reinforced Concrete Deck (design non-composite, detail composite)
2. Concrete Overlay, Grade E or C (non-composite)
3. Asphaltic Overlay with Waterproofing Membrane (not allowed)

19.3.9.3 Grout between Prestressed Box Girders

These sections are set 1" apart with a ±¼" tolerance. The space between sections is filled with a grout mix prior to post-tensioning the sections transversely. Post-tensioning is not allowed until the grout has cured for at least 48 hours and has attained a compressive strength of 3000psi.

19.4 Field Adjustments of Pretensioning Force

When strands are tensioned in open or unheated areas during cold weather they are subject to loss due to change in temperature. This loss can be compensated for by noting the change in temperature of the strands between tensioning and initial set of the concrete. For purposes of uniformity the strand temperature at initial concrete set is taken as 80°F.

Minor changes in temperature have negligible effects on the prestress force, therefore only at strand temperatures of 50°F and lower are increases in the tensioning force made.

Since plan prestress forces are based on 75% of the ultimate for low relaxation strands it is necessary to utilize the AASHTO allowable of temporarily overstressing up to 80% to provide for the losses associated with fabrication.

The following example outlines these losses and shows the elongation computations which are used in conjunction with jack pressure gages to control the tensioning of the strands.

Computation for Field Adjustment of Prestress Force

Known:

22 - 1/2", 7 wire low relaxation strands, A_{ps} = 0.1531 in^2

P_{pj} = 710.2 kips (jacking force from plan)

T_1 = 40°F (air temperature at strand tensioning)

T_2 = 80°F (concrete temperature at initial set)

L = 300' = 3,600" (distance from anchorage to reference point)

L_1 = 240' = 2,880" (length of cast segment)

E_p = 29,000 ksi (of prestressing tendons, sample tested from each spool)

C = 0.0000065 (coefficient of thermal expansion for steel, per degree F.)

COMPUTE:

jacking force per strand = P_{pj} = 710.2/22 = 32.3 kips

DL_1 = PL/AE = 32.3 x 3600/(0.1531 x 29,000) = 26.1"

Initial Load of 1.5 Kips to set the strands

DL_2 = 1.5 x 3600/(0.1531 x 29000) = 1.22"

DL_3 = Slippage in Strand Anchors = 0.45" (Past Experience)

DL_4 = Movement in Anchoring Abutments = 0.25" (Past Experience)

$DL_5 = C \times L_1 \times (T_2\text{-}T_1) = 0.0000065 \times 2880 \times 40 = 0.75''$

$P_{Loss} = DL_5 \times A \times E/L = 0.749 \times 0.1531 \times 29000/3600 = 0.9$ Kips

Total Prestress Force = $P + P_{Loss} = 32.3 + 0.9 = 33.2$ Kips

Total Elongation = $DL_1 + DL_3 + DL_4 + DL_5 = 27.55''$

Elongation After Initial Load = 27.55 - 1.22 = 26.33"

19.5 References

1. Whitney, C. S., "*Plastic Theory of Reinforced Concrete Design*", ASCE Trans., 107, 1942, p. 251.

2. Karr, P. H., Kriz, L. B. and Hognestad, E., "*Precast-Prestressed Concrete Bridges 1. Pilot Tests of Continuous Beams*", Portland Cement Association Development Department, Bulletin D34.

3. Mattock, A. H. and Karr, P. H., "*Precast-Prestressed Concrete Bridges 3. Further Tests of Continuous Girders*", Portland Cement Association Development Department, Bulletin D43.

4. Freyermuth, Clifford L., "*Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders (EB014.01E)*", Portland Cement Association, 1969.

5. Lin, T. Y. and Burns, N. H., "*Design of Prestressed Concrete Structures*", Third Edition, J. Wiley, 1981.

19.6 Design Examples

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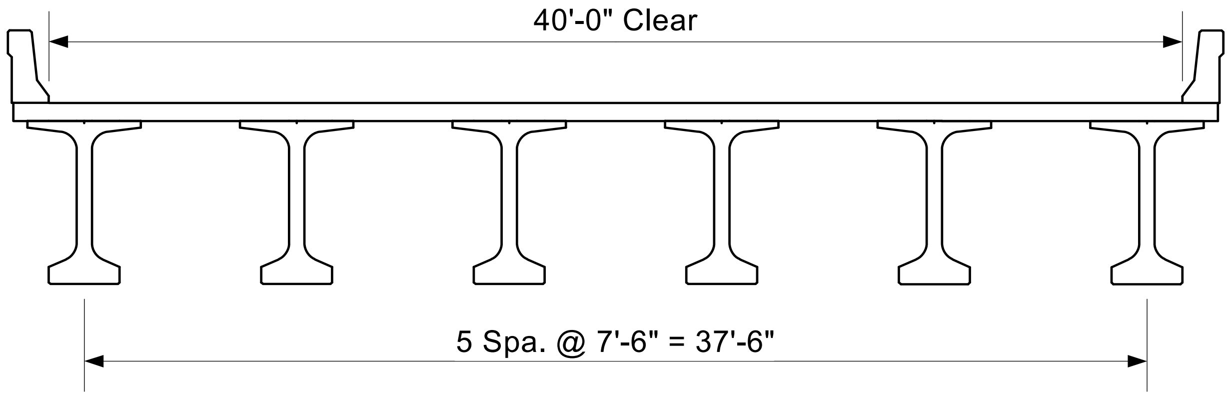
Table of Contents

E19-1 Single Span Bridge, 72W" Prestressed Girders - LRFD

This example shows design calculations for a single span prestressed gider bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

E19-1.1 Design Criteria



$L := 146$	center to center of bearing, ft
$L_g := 147$	total length of the girder (the girder extends 6 inches past the center of bearing at each abutment).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{ci} := 6.8$	girder initial concrete strength, ksi New limit for release strength.
$f'_{cd} := 4$	deck concrete strength, ksi
$f_{pu} := 270$	low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_s := 0.217$	area of strand, in^2
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$skew := 20$	skew angle, degrees
$E_s := 28500$	ksi, Modulus of Elasticity of the Prestressing Strands
$w_c := 0.150$	kcf

E19-1.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351} \qquad E_B := E_{beam8}$$

$$E_D := E_{deck4}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

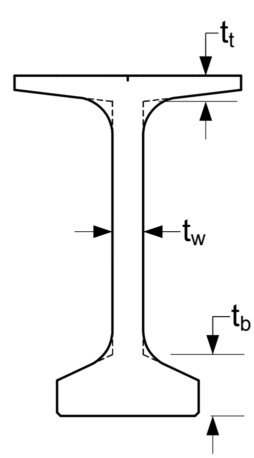
Note that this value of E_B is used for strength, composite section property, and long term deflection (deck and live load) calculations.

The value of the modulus of elasticity at the time of release is calculated in accordance with **LRFD [C5.4.2.4]**. This value of E_{ct} is used for loss and instantaneous deflection (due to prestress and dead load of the girder) calculations.

$$E_{beam6.8} := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_{ci}} \qquad \boxed{E_{beam6.8} = 4999} \qquad E_{ct} := E_{beam6.8}$$

E19-1.3 Section Properties

72W Girder Properties:



$w_{tf} := 48$ in

$t_t := 5.5$ in

$t_w := 6.5$ in

$t_b := 13$ in

$ht := 72$ in

$b_w := 30$ width of bottom flange, in

$A_g := 915$ in²

$r_{sq} := 717.5$ in²

$I_g := 656426$ in⁴

$y_t := 37.13$ in

$y_b := -34.87$ in

$S_t := 17680$ in³

$S_b := -18825$ in³

$e_g := y_t + 2 + \frac{t_{se}}{2}$ $\boxed{e_g = 42.88}$ in

Web Depth: $d_w := ht - t_t - t_b$ $\boxed{d_w = 53.50}$ in

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ **LRFD [Eq 4.6.2.2.1-1]** $\boxed{K_g = 3600866}$ in^4

E19-1.4 Girder Layout

Chapter 19 suggests that at a 146 foot span, the girder spacing should be 8'-6" with 72W girders.

$S := 8.5$ ft

Assume a minimum overhang of 2.5 feet (2 ft flange + 6" overhang), $s_{oh} := 2.5$

$n_{spa} := \frac{w_b - 2 \cdot s_{oh}}{S}$ $\boxed{n_{spa} = 4.412}$

Use the next lowest integer: $n_{spa} := \text{ceil}\left(n_{spa}\right)$ $\boxed{n_{spa} = 5}$

Number of girders: $ng := n_{spa} + 1$ $\boxed{ng = 6}$

Overhang Length: $s_{oh} := \frac{w_b - S \cdot n_{spa}}{2}$ $\boxed{s_{oh} = 0.00}$ ft

Recalculate the girder spacing based on a minimum overhang, $s_{oh} := 2.5$

$S := \frac{w_b - 2 \cdot s_{oh}}{n_{spa}}$ $\boxed{S = 7.50}$ ft

E19-1.5 Loads

$w_g := 0.953$ weight of 72W girders, klf

$w_d := 0.100$ weight of 8-inch deck slab (interior), ksf

$w_h := 0.125$ weight of 2.5-in haunch, klf

$w_{di} := 0.460$ weight of diaphragms on interior girder (assume 2), kips

$w_{dx} := 0.230$ weight of diaphragms on exterior girder, kips

$w_{ws} := 0.020$ future wearing surface, ksf

$w_p = 0.387$ weight of parapet, klf

E19-1.5.1 Dead Loads

Dead load on non-composite (DC):

exterior:

$$w_{dlxi} := w_g + w_d \cdot \left(\frac{S}{2} + s_{oh}\right) + w_h + 2 \cdot \frac{w_{dx}}{L} \qquad \boxed{w_{dlxi} = 1.706} \text{ klf}$$

interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L} \qquad \boxed{w_{dlii} = 1.834} \text{ klf}$$

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng} \qquad \boxed{w_p = 0.129} \text{ klf}$$

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng} \qquad \boxed{w_{ws} = 0.133} \text{ klf}$$

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E19-1.5.2 Live Loads

For Strength 1 and Service 1 and 3:

HL-93 loading = truck + lane **LRFD [3.6.1.3.1]**

tandem + lane

DLA of 33% applied to truck or tandem, but not to lane per **LRFD [3.6.2.1].**

For Fatigue:

LRFD [5.5.3] states that fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in **LRFD [Table 5.9.4.2.2-1]**.

For fully prestressed components, the compressive stress due to the Fatigue I load combination and one half the sum of effective prestress and permanent loads shall not exceed 0.40 f'c after losses.

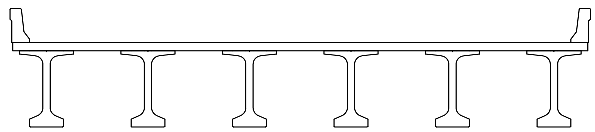
DLA of 15% applied to design truck with a 30 foot axle spacing.

For the Wisconsin Standard Permit Vehicle (Wis-250) Check:

The Wis-250 vehicle is to be checked during the design calculations to make sure it can carry a minimum vehicle weight of 190 kips. See Chapter 45 - Bridge Ratings for calculations.

E19-1.6 Load Distribution to Girders

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

Criteria for using distribution factors - Range of Applicability per **LRFD [Table 4.6.2.2.2b-1]**.

$$DeckSpan := \begin{array}{|l} \text{"OK"} \quad \text{if } 3.5 \le S \le 16 \\ \text{"NG"} \quad \text{otherwise} \end{array}$$

$$DeckThickness := \begin{array}{|l} \text{"OK"} \quad \text{if } 4.5 \le t_{se} \le 12 \\ \text{"NG"} \quad \text{otherwise} \end{array}$$

$$BridgeSpan := \begin{array}{|l} \text{"OK"} \quad \text{if } 20 \le L \le 240 \\ \text{"NG"} \quad \text{otherwise} \end{array}$$

$$NoBeams := \begin{array}{|l} \text{"OK"} \quad \text{if } ng \ge 4 \\ \text{"NG"} \quad \text{otherwise} \end{array}$$

$$LongitStiffness := \begin{array}{|l} \text{"OK"} \quad \text{if } 10000 \le K_g \le 7000000 \\ \text{"NG"} \quad \text{otherwise} \end{array}$$

$$x := \begin{pmatrix} S & DeckSpan \\ t_{se} & DeckThickness \\ L & BridgeSpan \\ ng & NoBeams \\ K_g & LongitStiffness \end{pmatrix}$$

$$x = \begin{pmatrix} 7.5 & \text{"OK"} \\ 7.5 & \text{"OK"} \\ 146.0 & \text{"OK"} \\ 6.0 & \text{"OK"} \\ 3600866.5 & \text{"OK"} \end{pmatrix}$$

E19-1.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\,3}}\right)^{0.1}$$

$$\boxed{g_{i1} = 0.435}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\,3}}\right)^{0.1}$$

$$\boxed{g_{i2} = 0.636}$$

$$g_i := \max\left(g_{i1}, g_{i2}\right)$$

$$\boxed{g_i = 0.636}$$

Note: The distribution factors above already have a multiple presence factor included that is used for service and strength limit states. For fatigue limit states, the 1.2 multiple presence factor should be divided out.

E19-1.6.2 Distribution Factors for Exterior Beams:

Two or More Lanes Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the following equations:

$$w_{parapet} := \frac{w_b - w}{2}$$

Width of parapet overlapping the deck

$$\boxed{w_{parapet} = 1.250} \text{ ft}$$

$$d_e := s_{oh} - w_{parapet}$$

Distance from the exterior web of exterior beam to the interior edge of parapet, ft.

$$\boxed{d_e = 1.250} \text{ ft}$$

Note: Conservatively taken as the distance from the center of the exterior girder.

Check range of applicability for de:

$d_{e\_check} := \begin{cases} \text{"OK"} & \text{if } -1.0 \le d_e \le 5.5 \\ \text{"NG"} & \text{otherwise} \end{cases}$ $\quad d_{e\_check} = \text{"OK"}$

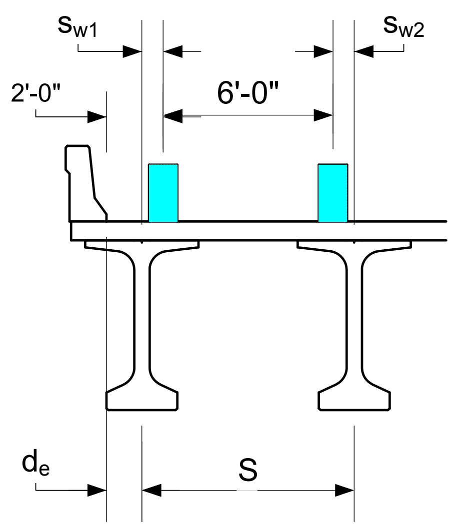
Note: While AASHTO allows the d_e value to be up to 5.5, the deck overhang (from the center of the exterior girder to the edge of the deck) is limited by WisDOT policy as stated in Chapter 17 of the Bridge Manual.

$e := 0.77 + \frac{d_e}{9.1}$ $\quad e = 0.907$

$g_{x2} := e \cdot g_i$ $\quad g_{x2} = 0.577$

One Lane Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the Lever Rule.



$s_{w1} := d_e - 2$ Distance from center of exterior girder to outside wheel load, ft. $\quad s_{w1} = -0.75$ ft

$s_{w2} := S + s_{w1} - 6$ Distance from wheel load to first interior girder, ft. $\quad s_{w2} = 0.75$ ft

$R_x := \frac{S + s_{w1} + s_{w2}}{S \cdot 2}$ $\quad R_x = 0.500$ % of a lane load

Add the single lane multi-presence factor, $m := 1.2$

$g_{x1} := R_x \cdot 1.2$ $\quad g_{x1} = 0.600$

The exterior girder distribution factor is the maximum value of the One Lane Loaded case and the Two or More Lanes Loaded case:

$g_x := \max(g_{x1}, g_{x2})$ $\boxed{g_x = 0.600}$

Note: The interior girder has a larger live load distribution factor and a larger dead load than the exterior girder. Therefore, for this example, the interior girder is likely to control.

E19-1.6.3 Distribution Factors for Fatigue:

The distribution factor for fatigue is the single lane distribution factor with the multi-presence factor, $m = 1.200$, removed:

$g_{if} := \dfrac{g_{i1}}{1.2}$ $\boxed{g_{if} = 0.362}$

E19-1.7 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in chapter 17 of this manual and as indicated below.

E19-1.7.1 Load Factors

From **LRFD [Table 3.4.1-1 & Table 3.4.1-4]**:

	DC	DW	LL
Strength 1	$\gamma st_{DC} := 1.25$	$\gamma st_{DW} := 1.50$	$\gamma st_{LL} := 1.75$
Service 1	$\gamma s1_{DC} := 1.0$	$\gamma s1_{DW} := 1.0$	$\gamma s1_{LL} := 1.0$
Service 3	$\gamma s3_{DC} := 1.0$	$\gamma s3_{DW} := 1.0$	$\gamma s3_{LL} := 0.8$ Check Tension Stress
Fatigue I			$\gamma f_{LL} := 1.50$

Dynamic Load Allowance (IM) is applied to the truck and tandem.

E19-1.7.2 Dead Load Moments

The unfactored dead load moments are listed below (values are in kip-ft):

Unfactored Dead Load Interior Girder Moments (kip-ft)				
	DC	DC	DC	DW
Tenth Point (Along Span)	girder at release	non-composite	composite	composite
0	35	0	0	0
0.1	949	1759	124	128
0.2	1660	3128	220	227
0.3	2168	4105	289	298
0.4	2473	4692	330	341
0.5	2574	4887	344	355

The DC_{nc} values are the component non-composite dead loads and include the weight of the girder, haunch, diaphragms and the deck.

The DC_c values are the component composite dead loads and include the weight of the parapets.

The DW_c values are the composite dead loads from the future wearing surface.

Note that the girder dead load moments at release are calculated based on the girder length. The moments for other loading conditions are calculated based on the span length (center to center of bearing).

E19-1.7.3 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the impact factor is applied only to the truck portion of the HL-93 loads. A separate analysis run will be required if results without impact are desired.

Unfactored Live Load + Impact Moments per Lane (kip-ft)			
Tenth Point	Truck	Tandem	Fatigue
0	0	0	0
0.1	1783	1474	937
0.2	2710	2618	1633
0.3	4100	3431	2118
0.4	4665	3914	2383
0.5	4828	4066	2406

The Wisconsin Standard Permit Vehicle should also be checked. See Chapter 45 - Bridge Rating for further information.

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$g_i = 0.636$

$M_{LL} = g_i \cdot 4828$ $\boxed{M_{LL} = 3073}$ kip-ft

$\boxed{g_{if} = 0.362}$

$M_{LLfat} := g_{if} \cdot 2406$ $\boxed{M_{LLfat} = 871}$ kip-ft

E19-1.7.4 Factored Moments

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored moments for each limit state are calculated by applying the appropriate load factor to the girder moments. For the interior girder:

Strength 1

$$M_{str} := \eta \cdot \left[\gamma st_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma st_{DW} \cdot M_{DWc} + \gamma st_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.25 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.50 \cdot M_{DWc} + 1.75 \cdot M_{LL}\right] \qquad \boxed{M_{str} = 12449} \text{ kip-ft}$$

Service 1 (for compression checks)

$$M_{s1} := \eta \cdot \left[\gamma s1_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma s1_{DW} \cdot M_{DWc} + \gamma s1_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 1.0 \cdot M_{LL}\right] \qquad \boxed{M_{s1} = 8659} \text{ kip-ft}$$

Service 3 (for tension checks)

$$M_{s3} := \eta \cdot \left[\gamma s3_{DC} \cdot \left(M_{DLnc} + M_{DLc}\right) + \gamma s3_{DW} \cdot M_{DWc} + \gamma s3_{LL} \cdot M_{LL}\right]$$

$$= 1.0 \cdot \left[1.0 \cdot \left(M_{DLnc} + M_{DLc}\right) + 1.0 \cdot M_{DWc} + 0.8 \cdot M_{LL}\right] \qquad \boxed{M_{s3} = 8045} \text{ kip-ft}$$

Service 1 and 3 non-composite DL alone

$$M_{nc} := \eta \cdot \gamma s1_{DC} \cdot M_{DLnc} \qquad \boxed{M_{nc} = 4887} \text{ kip-ft}$$

Fatigue 1

$$M_{fat} := \eta \cdot \gamma f_{LL} \cdot M_{LLfat} \qquad \boxed{M_{fat} = 1307} \text{ kip-ft}$$

E19-1.8 Composite Girder Section Properties

Calculate the effective flange width in accordance with **LRFD [4.6.2.6]** and section 17.2.11 of the Wisconsin Bridge Manual:

$w_e := S \cdot 12$ $\boxed{w_e = 90.00}$ in

The effective width, w_e, must be adjusted by the modular ratio, n, to convert to the same concrete material (modulus) as the girder.

$w_{eadj} := \dfrac{w_e}{n}$ $\boxed{w_{eadj} = 58.46}$ in

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

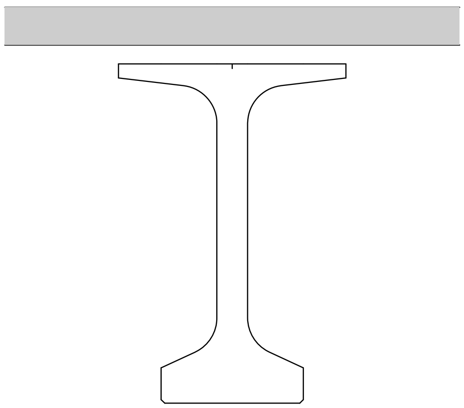
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

haunch thickness; $hau := 2.00$ in

total height; $h_c := ht + hau + t_{se}$

$\boxed{h_c = 81.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	77.75	438	34088	2650309	2055	2652364
Girder	34.87	915	31906	1112564	656426	1768990
Haunch	73	0	0	0	0	0
Summation		1353	65994			4421354

$\boxed{\Sigma A = 1353}$ in^2

$\boxed{\Sigma AY = 65994}$ in^3

$\boxed{\Sigma IplusAYsq = 4421354}$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$$ $\boxed{y_{cgb} = -48.8}$ in

$$y_{cgt} := ht + y_{cgb}$$ $\boxed{y_{cgt} = 23.2}$ in

$$A_{cg} := \Sigma A$$ $\boxed{A_{cg} = 1353}$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^2$$ $\boxed{I_{cg} = 1203475}$ in^4

$$S_{cgt} := \frac{I_{cg}}{y_{cgt}}$$ $\boxed{S_{cgt} = 51786}$ in^3

$$S_{cgb} := \frac{I_{cg}}{y_{cgb}}$$ $\boxed{S_{cgb} = -24681}$ in^3

Deck:

$$S_{cgdt} := n \cdot \frac{I_{cg}}{y_{cgt} + hau + t_{se}}$$ $\boxed{S_{cgdt} = 56594}$ in^3

$$S_{cgdb} := n \cdot \frac{I_{cg}}{y_{cgt} + hau}$$ $\boxed{S_{cgdb} = 73411}$ in^3

E19-1.9 Preliminary Design Information:

Controlling Design Criteria

A: At transfer, precasting plant:

T is maximum, little loading
Load = $T_{initial}$ (before losses) + M_g (due to girder weight)

Avoid high initial tension or compression with initial concrete strength.

B: At full service load, final loading (say after 50 years):

T is minimum, load is max
Load = $T_{initial}$ (before losses) + M_g (max service moment)

Avoid cracking and limit concrete stress.

At transfer (Interior Girder):

$M_{iend} := 0$ kip-ft

$M_g := w_g \cdot \frac{L_g^{\,2}}{8}$ $\boxed{M_g = 2574}$ kip-ft

After 50 Years (Interior Girder):

Service 1 Moment $\boxed{M_{s1} = 8659}$ kip-ft

Service 3 Moment $\boxed{M_{s3} = 8045}$ kip-ft

Service 1 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{1c} := M_{s1} - M_{nc}$ $\boxed{M_{1c} = 3772}$ kip-ft

Service 3 Moment Components:

non-composite moment (girder + deck) $\boxed{M_{nc} = 4887}$ kip-ft

composite moment (parapet, FWS and LL)

$M_{3c} := M_{s3} - M_{nc}$ $\boxed{M_{3c} = 3157}$ kip-ft

At 50 years the prestress has decreased (due to CR, SH, RE):

The approximate method of estimated time dependent losses is used by WisDOT. The lump sum loss estimate, I-girder loss **LRFD [5.9.5.3]**

Where PPR is the partial prestressing ratio, $PPR := 1.0$

$F_{delta} := 33 \cdot \left(1 - 0.15 \cdot \frac{f'_c - 6}{6}\right) + 6 \cdot PPR$ $\boxed{F_{delta} = 37.350}$ ksi

but, for low relaxation strand: $F_{Delta} := F_{delta} - 6$ $\boxed{F_{Delta} = 31.350}$ ksi

Assume an initial strand stress; $f_{tr} := 0.75 \cdot f_{pu}$ $\boxed{f_{tr} = 202.500}$ ksi

Based on experience, assume $\Delta f_{pES\_est} := 18$ ksi loss from elastic shortening. As an alternate initial estimate, **LRFD [C.5.9.5.2.3a]** suggests assuming a 10% ES loss.

$$ES_{loss} := \frac{\Delta f_{pES\_est}}{f_{tr}} \cdot 100 \qquad \boxed{ES_{loss} = 8.889} \ \%$$

$$f_i := f_{tr} - \Delta f_{pES\_est} \qquad \boxed{f_i = 184.500} \ \text{ksi}$$

The total loss is the time dependant losses plus the ES losses:

$$loss := F_{Delta} + \Delta f_{pES\_est} \qquad \boxed{loss = 49.350} \ \text{ksi}$$

$$loss_{\%} := \frac{loss}{f_{tr}} \cdot 100 \qquad \boxed{loss_{\%} = 24.370} \ \% \text{ (estimated)}$$

If T_o is the initial prestress, then (1-loss)*T_o is the remaining:

$$T = \left(1 - loss_{\%}\right) \cdot T_o$$

$$ratio := 1 - \frac{loss_{\%}}{100} \qquad \boxed{ratio = 0.756}$$

$$T = ratio \cdot T_o$$

E19-1.10 Preliminary Design Steps

The following steps are utilized to design the prestressing strands:

1) Design the amount of prestress to prevent tension at the bottom of the beam under the full load at center span after 50 years.

2) Calculate the prestress losses and check the girder stresses at mid span at the time of transfer.

3) Design the eccentricity of the strands at the girder end to avoid tension or compression over-stress at the time of transfer.

4) If required, design debonding of strands to prevent over-stress at the girder ends.

5) Check resulting stresses at the critical sections of the girder at the time of transfer and after 50 years.

E19-1.10.1 Determine Amount of Prestress

Design the amount of prestress to prevent tension at the bottom of the beam under the full loac (at center span) after 50 years.

Near center span, after 50 years, T = the remaining effective prestress, aim for no tension at the bottom. Use Service I for compression and Service III for tension.

For this example, the interior girder has the controlling moments.

Calculate the stress at the bottom of the beam due to combination of non-composite and composite loading (Service 3 condition):

$$f_b := \frac{M_{nc} \cdot 12}{S_b} + \frac{M_{3c} \cdot 12}{S_{cgb}} \qquad \boxed{f_b = -4.651} \text{ ksi}$$

Stress at bottom due to prestressing (after losses):

$$f_{bp} = \frac{T}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right) \qquad \text{where } T = \left(1 - loss_{\%}\right) \cdot T_o$$

and $f_{bp} := -f_b$ desired final prestress.

We want this to balance out the tensile stress calculated above from the loading, i.e. an initial compression. Since we are making some assumptions on the actual losses, we are ignoring the allowable tensile stress in the concrete for these calculations.

$$f_{bp} = \frac{\left(1 - loss_{\%}\right) \cdot T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right) \qquad \text{(after losses)}$$

OR:

$$\frac{f_{bp}}{1 - loss_{\%}} = \frac{T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

$$f_{bpi\_1} := \frac{f_{bp}}{1 - \frac{loss_{\%}}{100}}$$ $\boxed{f_{bpi\_1} = 6.149}$ ksi

desired bottom initial prestress (before losses)

If we use the actual allowable tensile stress in the concrete, the desired bottom initial prestress is calculated as follows:

The allowable tension, from **LRFD [5.9.4.2.2]**, is:

$f_{tall} := 0.19 \cdot \lambda \sqrt{f'_c} \leq 0.6$ ksi; $\lambda = 1.0$ (norm. wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tall} = 0.537}$ ksi

The desired bottom initial prestress (before losses):

$f_{bpi\_2} := f_{bpi\_1} - f_{tall}$ $\boxed{f_{bpi\_2} = 5.612}$ ksi

Determine the stress effects for different strand patterns on the 72W girder:

$A_s = 0.217$ in^2

$f'_s := 270000$ psi

$f_s := 0.75 \cdot f'_s$ $\boxed{f_s = 202500}$ psi

$P := A_s \cdot \frac{f_s}{1000}$ $\boxed{P = 43.943}$ kips

$$f_{bpi} := \frac{P \cdot N}{A_g} \cdot \left(1 + e \cdot \frac{y_b}{r_{sq}}\right)$$ (bottom initial prestress - before losses)

The values of f_{bpi} for various strand patterns is shown in the following table.

72W Stress Effects		
Pi (per strand) = 43.94 kips		
No. Strands	e (in)	bottom stress (ksi)
36	-31.09	4.3411
38	-30.98	4.5726
40	-30.87	4.8030
42	-30.77	5.0333
44	-30.69	5.2648
46	-30.52	5.4858
48	-30.37	5.7075
50	-30.23	5.9290
52	-30.10	6.1504

Solution:
Try $ns := 46$ strands, 0.6 inch diameter.
Initial prestress at bottom $f_{bpi} := 5.4858$ ksi,
Eccentricity, $e_s := -30.52$ inches; actual tension should be less than allowed.

E19-1.10.2 Prestress Loss Calculations

The loss in prestressing force is comprised of the following components:

1) Elastic Shortening (ES), shortening of the beam as soon as prestress is applied.

2) Shrinkage (SH), shortening of the concrete as it hardens, time function.

3) Creep (CR), slow shortening of concrete due to permanent compression stresses in the beam, time function.

4) Relaxation (RE), the tendon slowly accommodates itself to the stretch and the internal stress drops with time

E19-1.10.2.1 Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$T_{oi} := ns \cdot f_{tr} \cdot A_s$ $\boxed{= 46 \cdot 0.75 \cdot 270 \cdot 0.217 = 2021}$ kips

The ES loss estimated above was: $\Delta f_{pES\_est} = 18.0$ ksi, or $ES_{loss} = 8.889$ %. The resulting force in the strands after ES loss:

$$T_o := \left(1 - \frac{ES_{loss}}{100}\right) \cdot T_{oi}$$ $\boxed{T_o = 1842}$ kips

If we assume all strands are straight we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A_g} + (T_o \cdot e_s) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g}$$ $\boxed{f_{cgp} = 3.190}$ ksi

$\boxed{E_{ct} = 4999}$ ksi

$E_p := E_s$ $\boxed{E_p = 28500}$ ksi

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp}$$ $\boxed{\Delta f_{pES} = 18.185}$ ksi

This value of Δf_{pES} is in agreement with the estimated value above; $\Delta f_{pES\_est} = 18.00$ ksi. If these values did not agree, T_o would have to be recalculated using f_{tr} minus the new value of Δf_{pES}, and a new value of f_{cgp} would be determined. This iteration would continue until the assumed and calculated values of Δf_{pES} are in agreement.

The initial stress in the strand is:

$f_i := f_{tr} - \Delta f_{pES}$ $\boxed{f_i = 184.315}$ ksi

The force in the beam after transfer is:

$T_o := ns \cdot A_s \cdot f_i$ $\boxed{T_o = 1840}$ kips

Check the design to avoid premature failure at the center of the span at the time of transfer. Check the stress at the center span (at the plant) at both the top and bottom of the girder.

$$f_{ttr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$ $\boxed{f_{ttr} = 0.582}$ ksi

$$f_{btr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$ $\boxed{f_{btr} = 3.353}$ ksi

temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

$f_{ciall} := 0.65 \cdot f'_{ci}$ $\boxed{f_{ciall} = 4.420}$ ksi

Is the stress at the bottom of the girder less than the allowable? $\boxed{check = \text{"OK"}}$

E19-1.10.2.2 Approximate Estimate of Time Dependant Losses

Calculate the components of the time dependant losses; shrinkage, creep and relaxation, using the approximate method in accordance with **LRFD [5.9.5.3]**.

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pi} \cdot A_s}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

From **LRFD [Figure 5.4.2.3.3-1]**, the average annual ambient relative humidity, $H := 72$ %.

$\gamma_h := 1.7 - 0.01 \cdot H$ $\boxed{\gamma_h = 0.980}$

$$\gamma_{st} := \frac{5}{1 + f'_{ci}}$$ $\boxed{\gamma_{st} = 0.641}$

$\Delta f_{pR} := 2.4$ ksi for low relaxation strands

$$\Delta f_{pCR} := 10.0 \cdot \frac{f_{tr} \cdot A_s \cdot ns}{A_g} \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pCR} = 13.878}$ ksi

$$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pSR} = 7.538}$ ksi

$$\Delta f_{pRE} := \Delta f_{pR}$$ $\boxed{\Delta f_{pRE} = 2.400}$ ksi

$$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$$ $\boxed{\Delta f_{pLT} = 23.816}$ ksi

The total estimated prestress loss (Approximate Method):

$$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$$ $\boxed{\Delta f_p = 42.001}$ ksi

$\boxed{\frac{\Delta f_p}{f_{tr}} \cdot 100 = 20.741}$ % total prestress loss

This value is slightly less than but in general agreement with the initial estimated $loss_{\%} = 24.370$.

The remaining stress in the strands and total force in the beam after all losses is:

$$f_{pe} := f_{tr} - \Delta f_p$$ $\boxed{f_{pe} = 160.50}$ ksi

$$T := ns \cdot A_s \cdot f_{pe}$$ $\boxed{T = 1602}$ kips

E19-1.10.3 Design of Strand Drape

Design the eccentricity of the strands at the end to avoid tension or compression over stress at the time of transfer. Check the top stress at the end. If the strands are straight, $M_g = 0$.

top:

$$f_{tetr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t}$$ $\boxed{f_{tetr} = -1.165}$ ksi

high tension stress

In accordance with **LRFD Table [5.9.4.1.2-1]**, the temporary allowable tension stress is calculated as follows (assume there is no bonded reinforcement):

$f_{tiall} := -\min\left(0.0948 \cdot \lambda \sqrt{f'_{ci}}, 0.2\right)$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tiall} = -0.200}$ ksi

bottom:

$$f_{betr} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b}$$

$\boxed{f_{betr} = 4.994}$ ksi

$\boxed{f_{ciall} = 4.420}$ ksi

high compressive stress

The tension at the top is too high, and the compression at the bottom is also too high!!

Drape some of the strands upward to decrease the top tension and decrease the compression at the bottom.

Find the required position of the steel centroid to avoid tension at the top. Conservatively set the top stress equal to zero and solve for "e":

$$f_{tetr} = \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t}$$

$$e_{sendt} := \frac{S_t}{T_o} \cdot \left(0 - \frac{T_o}{A_g}\right)$$

$\boxed{e_{sendt} = -19.32}$ inches or higher

Therefore, we need to move the resultant centroid of the strands up:

$move := e_{sendt} - e_s$ $\boxed{move = 11.20}$ inches upward

Find the required position of the steel centroid to avoid high compression at the bottom of the beam. Set the bottom compression equal to the allowable stress and find where the centroid of $ns = 46$ strands needs to be:

$$f_{betr} = \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b}$$

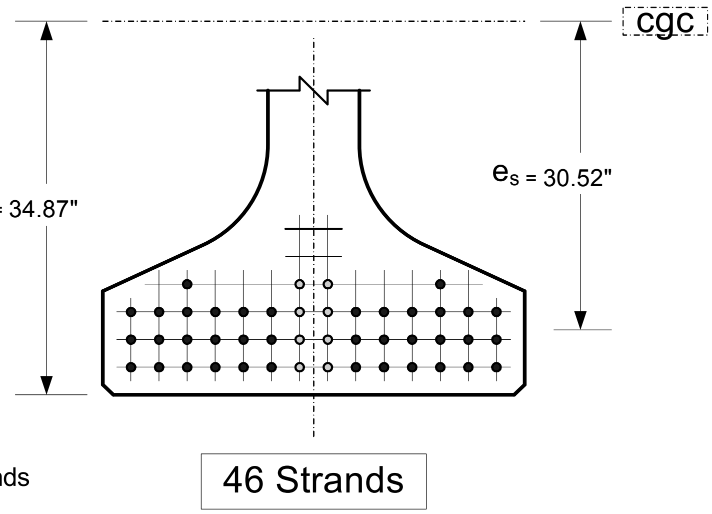
Set equal to allowed: $f_{betr} := f_{ciall}$

$$e_{sendb} := \frac{S_b}{T_o} \cdot \left(f_{ciall} - \frac{T_o}{A_g}\right)$$

$\boxed{e_{sendb} = -24.65}$ inches or higher

Top stress condition controls:

$e_{send} := \max\left(e_{sendt}, e_{sendb}\right)$ $\boxed{e_{send} = -19.32}$ inches



38 undraped strands
8 draped strands

LRFD [Table 5.12.3-1] requires 2 inches of cover. However, WisDOT uses 2 inches to the center of the strand, and 2 inch spacing between centers.

The center $ns_d := 8$ strands will be draped at the end of the girder.

Find the center of gravity of the remaining $ns_s = 38$ straight strands from the bottom of the girder:

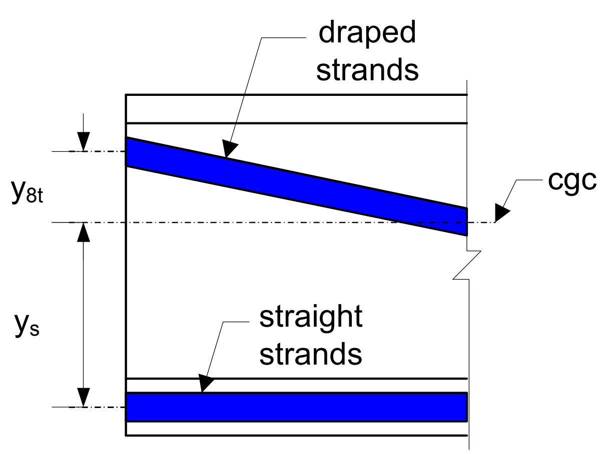
$$Y_s := \frac{12 \cdot 2 + 12 \cdot 4 + 12 \cdot 6 + 2 \cdot 8}{ns_s}$$

$\boxed{Y_s = 4.21}$ inches from the bottom of the girder

OR:

$$y_s := y_b + Y_s$$

$\boxed{y_s = -30.66}$ inches from the center of gravity of the girder (cgc)



y_{8t} is the eccentricity of the draped strands at the end of the beam. We want the eccentricity of all of the strands at the end of the girder to equal, $e_{send} = -19.322$ inches for stress control.

$$e_{send} = \frac{ns_s \cdot y_s + ns_d \cdot y_{8t}}{ns}$$

$$y_{8t} := \frac{ns \cdot e_{send} - ns_s \cdot y_s}{ns_d}$$ $\boxed{y_{8t} = 34.53}$ inches above the cgc

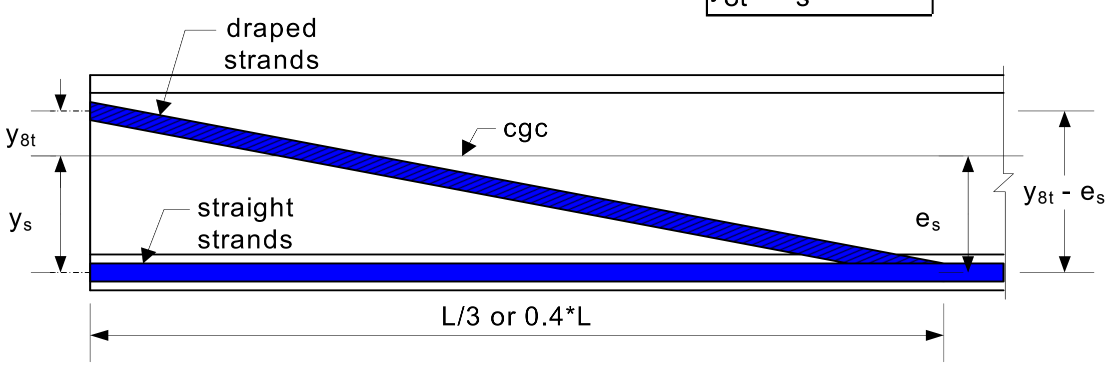
However, $y_t = 37.13$ inches to the top of the beam. If the draped strands are raised $y_{8t} = 34.53$ inches or more above the cgc, the stress will be OK.

Drape the center strands the maximum amount: Maximum drape for $ns_d = 8$ strands:

$y_{8t} := y_t - 5$ $\boxed{y_{8t} = 32.13}$ in

$\boxed{e_s = -30.52}$ in

$\boxed{y_{8t} - e_s = 62.65}$ in



Try a drape length of: $\frac{L_g}{3} = 49.00$ feet $HD := \frac{L_g}{3}$

The eccentricity of the draped strands at the hold down point:

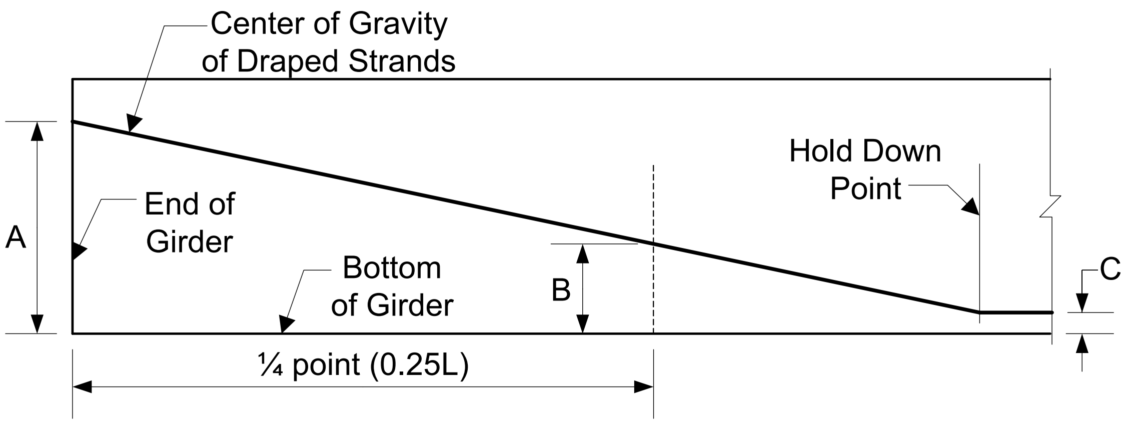
$e_{8hd} := y_b + 5$ $\boxed{e_{8hd} = -29.870}$ in

Strand slope, $slope := \dfrac{y_{8t} - e_{8hd}}{(HD \cdot 12)} \cdot 100$ $\boxed{slope = 10.54}$ %

Is the slope of the strands less than 12%? $\boxed{check = \text{"OK"}}$

12% is a suggested maximum slope, actual acceptable slope is dependant on the form capacity or on the fabricator.

Calculate the values of A, B_{min}, B_{max} and C to show on the plans:



$A := |y_b| + y_{8t}$ $\boxed{A = 67.00}$ in

$\boxed{C := 5.00}$ in

$B_{min} := \dfrac{A + 3C}{4}$ $\boxed{B_{min} = 20.50}$ in

$B_{max} := B_{min} + 3$ $\boxed{B_{max} = 23.50}$ in

Check hold down location for B_{max} to make sure it is located between $L_g/3$ and $0.4 {*} L_g$:

$slope_{Bmax} := \dfrac{A - B_{max}}{0.25 \cdot L_g \cdot 12}$ $\boxed{slope_{Bmax} = 0.099}$ ft/ft

$x_{Bmax} := \dfrac{A - C}{slope_{Bmax}} \cdot \dfrac{1}{12}$ $\boxed{x_{Bmax} = 52.38}$ ft

$\boxed{L_g \cdot 0.4 = 58.80}$ ft

Is the resulting hold down location less than $0.4 {*} L_g$? $\boxed{check = \text{"OK"}}$

Check the girder stresses at the end of the transfer length of the strands at release:

Minimum moment on section = girder moment at the plant

The transfer length may be taken as:

$l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

$x := \frac{l_{tr}}{12}$ $\boxed{x = 3.00}$ feet

The eccentricity of the draped strands and the entire strand group at the transfer length is:

$y_{8tt} := y_{8t} - \frac{slope}{100} \cdot x \cdot 12$ $\boxed{y_{8tt} = 28.334}$ in

$e_{st} := \frac{ns_s \cdot y_s + 8 \cdot y_{8tt}}{ns}$ $\boxed{e_{st} = -20.400}$ in

The moment at the end of the transfer length due to the girder dead load:

$M_{gt} := \frac{w_g}{2} \cdot \left(L_g \cdot x - x^2\right)$ $\boxed{M_{gt} = 206}$ kip-ft

The girder stresses at the end of the transfer length:

$f_{tt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$ $\boxed{f_{tt} = 0.028}$ ksi

$\boxed{f_{tiall} = -0.200}$ ksi

Is f_{tt} less than f_{tiall}? $\boxed{check = \text{"OK"}}$

$f_{bt} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$ $\boxed{f_{bt} = 3.873}$ ksi

$\boxed{f_{ciall} = 4.420}$ ksi

Is f_{bt} less than f_{ciall}? $\boxed{check = \text{"OK"}}$

E19-1.10.4 Stress Checks at Critical Sections

Critical Sections	Critical Conditions		
	At Transfer	Final	Fatigue
Girder Ends	X		
Midspan	X	X	X
Hold Down Points	X	X	X

Data:

$T_o = 1840$ kips $\quad$ $T = 1602$ kips

$M_{nc} = 4887$ kip-ft $\quad$ $M_{s3} = 8045$ kip-ft

$M_{s1} = 8659$ kip-ft $\quad$ $M_g = 2574$ kip-ft

Need moments at hold down points: $\frac{L_g}{3} = 49.00$ feet, from the end of the girder.

girder: $M_{ghd} = 2288$ kip-ft

non-composite: $M_{nchd} = 4337$ kip-ft

Service I composite: $M_{1chd} = 3371$ kip-ft

Service III composite: $M_{3chd} = 2821$ kip-ft

Note: The release girder moments shown above at the hold down location are calculated based on the total girder length.

Check the girder at the end of the beam (at the transfer length):

$e_{st} = -20.40$ inches $\quad$ $f_{tiall} = -0.200$ ksi $\quad$ $f_{ciall} = 4.420$ ksi

At transfer, $M_{gt} = 206$ kip-ft

Top of girder (Service 3):

$$f_{tei} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$$

$f_{tei} = 0.028$ ksi

Is f_{tei} greater than f_{tiall}? check = "OK"

Bottom of girder (Service 1):

$$f_{bei} := \frac{T_o}{A_g} + \frac{T_o \cdot e_{st}}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$$

$f_{bei} = 3.873$ ksi

Is f_{bei} less than f_{ciall}? check = "OK"

Check at the girder and deck at midspan:

$\boxed{e_s = -30.52}$ inches

Initial condition at transfer: $\boxed{f_{tiall} = -0.200}$ ksi $\boxed{f_{ciall} = 4.420}$ ksi

Top of girder stress (Service 3):

$$f_{ti} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$ $\boxed{f_{ti} = 0.582}$ ksi

Is f_{ti} greater than f_{tiall}? $\boxed{check = \text{"OK"}}$

Bottom of girder stress (Service 1):

$$f_{bi} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$ $\boxed{f_{bi} = 3.353}$ ksi

Is f_{bi} less than f_{ciall}? $\boxed{check = \text{"OK"}}$

Final condition:

Allowable Stresses,**LRFD [5.9.4.2]**:

There are two compressive stress limits: (Service 1) **LRFD [5.9.4.2.1]**

$f_{call1} := 0.45 \cdot f'_c$ PS + DL $\boxed{f_{call1} = 3.600}$ ksi

$f_{call2} := 0.60 \cdot f'_c$ LL + PS + DL $\boxed{f_{call2} = 4.800}$ ksi

(Service 3) **LRFD [5.9.4.2.2]** (Moderate Corrosion Condition)

tension: $f_{tall} = -0.19 \cdot \lambda \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$f_{tall} := -0.19 \cdot \sqrt{f'_c}$ $|f_{tall}| \le 0.6$ ksi $\boxed{f_{tall} = -0.537}$ ksi

Allowable Stresses (Fatigue),**LRFD [5.5.3]**:

Fatige compressive stress limit:

$f_{call\_fat} := 0.40 \cdot f'_c$ LLfat + 1/2(PS + DL) $\boxed{f_{call\_fat} = 3.200}$ ksi

Top of girder stress (Service 1):

$$f_{t1} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{(M_{DLc} + M_{DWc}) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t1} = 2.465}$ ksi

$$f_{t2} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{(M_{DLc} + M_{DWc} + M_{LL}) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t2} = 3.177}$ ksi

Is f_t less than f_{call}?

$\boxed{\text{check1} = \text{"OK"}}$

$\boxed{\text{check2} = \text{"OK"}}$

Top of girder stress (Fatigue 1):

$$f_{tfat} := \frac{1}{2}\left(\frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t}\right) + \frac{\left[\frac{1}{2}(M_{DLc} + M_{DWc}) + M_{LLfat}\right] \cdot 12}{S_{cgt}}$$

$\boxed{f_{tfat} = 1.434}$ ksi

Is f_{tfat} less than f_{call_fat}?

$\boxed{\text{check} = \text{"OK"}}$

Bottom of girder stress (Service 3):

$$f_b := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b} + \frac{M_{nc} \cdot 12}{S_b} + \frac{(M_{s3} - M_{nc}) \cdot 12}{S_{cgb}}$$

$\boxed{f_b = -0.302}$ ksi

Is f_{tb} greater than f_{tall}?

$\boxed{\text{check} = \text{"OK"}}$

Top of deck stress (Service 1):

$$f_{dall} := 0.40 \cdot f'_{cd}$$

$\boxed{f_{dall} = 1.600}$ ksi

$$f_{dt} := \frac{(M_{s1} - M_{nc}) \cdot 12}{S_{cgdt}}$$ $\boxed{f_{dt} = 0.800}$ ksi

Is f_{dt} less than f_{dall}? $\boxed{check = \text{"OK"}}$

Bottom of deck stress (Service 1):

$$f_{db} := \frac{(M_{s1} - M_{nc}) \cdot 12}{S_{cgdb}}$$ $\boxed{f_{db} = 0.617}$ ksi

Is f_{db} less than f_{dall}? $\boxed{check = \text{"OK"}}$

Check at hold-down location:

At transfer: $\boxed{f_{tiall} = -0.200}$ ksi $\boxed{f_{ciall} = 4.420}$ ksi

Top of girder stress (Service 3):

$$f_{t3i} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{ghd} \cdot 12}{S_t}$$ $\boxed{f_{t3i} = 0.388}$ ksi

Is f_{t3i} greater than f_{tiall}? $\boxed{check = \text{"OK"}}$

Bottom of girder stress (Service 1):

$$f_{b3i} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{ghd} \cdot 12}{S_b}$$ $\boxed{f_{b3i} = 3.535}$ ksi

Is f_{b3i} less than f_{ciall}? $\boxed{check = \text{"OK"}}$

Final condition, after 50 years, full load: $\boxed{f_{tall} = -0.537}$ ksi $\boxed{f_{call2} = 4.800}$ ksi

Top of girder stress (Service 1):

$$f_{t3} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{nchd} \cdot 12}{S_t} + \frac{M_{1chd} \cdot 12}{S_{cgt}}$$ $\boxed{f_{t3} = 2.710}$ ksi

Is f_{t3} less than f_{call2}? $\boxed{check = \text{"OK"}}$

Top of girder stress (Fatigue 1):

$$f_{tfat} := \frac{1}{2} \cdot \left(\frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{nchd} \cdot 12}{S_t} \right) + \frac{M_{fatchd} \cdot 12}{S_{cgt}}$$ $\boxed{f_{tfat} = 1.317}$ ksi

Is f_{tfat} less than f_{call_fat}? $\boxed{check = "OK"}$

Bottom of girder stress (Service 3):

$$f_{b3} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b} + \frac{M_{nchd} \cdot 12}{S_b} + \frac{M_{3chd} \cdot 12}{S_{cgb}}$$ $\boxed{f_{b3} = 0.212}$ ksi

Is f_{b3} greater than f_{tall}? $\boxed{check = "OK"}$

Top of deck stress (Service 1): $\boxed{f_{dall} = 1.600}$ ksi

$$f_{dt3} := \frac{(M_{1chd}) \cdot 12}{S_{cgdt}}$$ $\boxed{f_{dt3} = 0.715}$ ksi

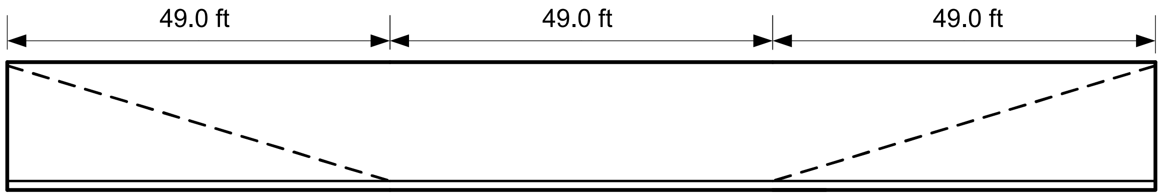
Is f_{dt} less than f_{dall}? $\boxed{check = "OK"}$

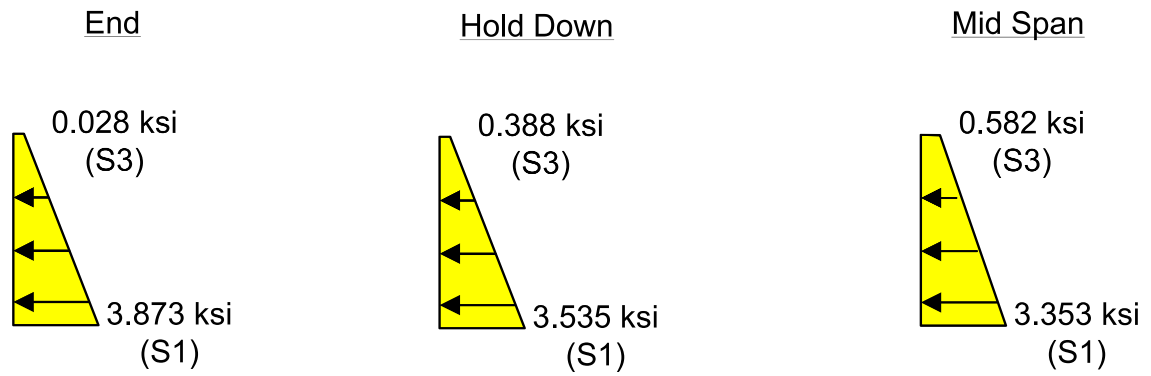
Bottom of deck stress (Service 1):

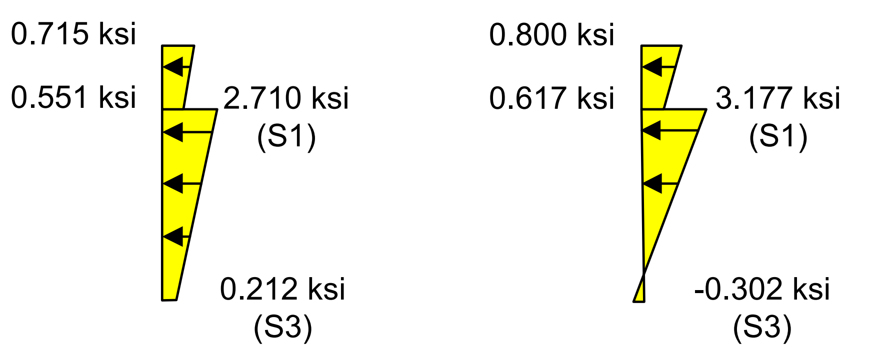
$$f_{db3} := \frac{(M_{1chd}) \cdot 12}{S_{cgdb}}$$ $\boxed{f_{db3} = 0.551}$ ksi

Is f_{db} less than f_{dall}? $\boxed{check = "OK"}$

Summary of Design Stresses:







Initial Allowable:

$\text{compression} := f_{ciall} = 4.42$ ksi

Final Allowable:

$\text{compression}_1 := f_{call1} = 3.6$ ksi

$\text{compression}_2 := f_{call2} = 4.8$ ksi

$\text{compression\_fatigue} := f_{call\_fat} = 3.2$ ksi

$\text{tension} = f_{tall} = -0.537$ ksi **All stresses are acceptable!**

E19-1.11 Calculate Jacking Stress

The fabricator is responsible for calculation of the jacking force. See **LRFD [5.9.3]** for equations for low relaxation strands.

E19-1.12 Flexural Strength Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.7.3.1.1]**:

$\boxed{f_{pe} = 160}$ ksi $\boxed{0.5 \cdot f_{pu} = 135}$ ksi

Is 0.5*f_{pu} less than f_{pe}? $\boxed{check = \text{"OK"}}$

Then at failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

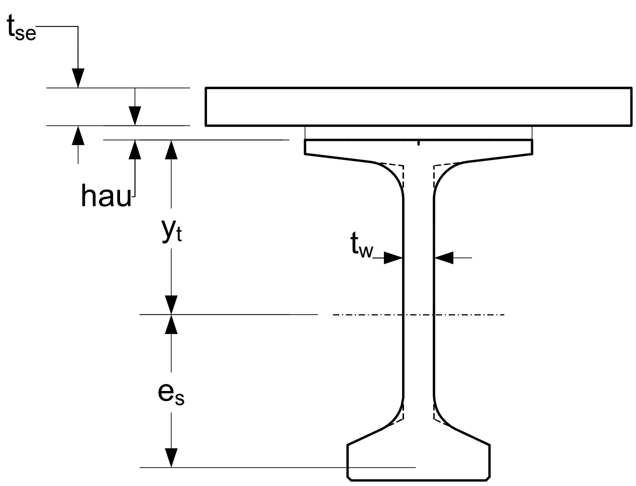
where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD Table [C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assumed dimensions:



Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}}$$

where:

$A_{ps} := ns \cdot A_s$ $\boxed{A_{ps} = 9.98}$ in^2

$b := w_e$ $\boxed{b = 90.00}$ in

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \leq 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_{cd} - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.850}$

$d_p := y_t + hau + t_{se} - e_s$ $\boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.99}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.49}$ in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$ depth of compression flange $\boxed{h_f = 7.500}$ in

$w_{tf} = 48.00$ width of top flange, inches

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.937}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 9.30}$ in

This is within the depth of the haunch (9.5 inches). Therfore our assumption is OK.

Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 259.283}$ ksi

$T_u := f_{ps} \cdot A_{ps}$ $\boxed{T_u = 2588}$ kips

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.7.3.2]; [5.7.3.2.2]**

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 15717}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\quad\boxed{M_r = 15717}$ kip-ft

The required capacity:

Interior Girder Moment $\quad\boxed{M_{str} = 12449}$ kip-ft

Exterior Girder Moment $\quad\boxed{M_{strx} = 11183}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]** for the interior girder:

$\boxed{1.33 \cdot M_{str} = 16558}$ kip-ft

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\quad\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]** $\quad\boxed{f_r = 0.679}$ ksi

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 4.348}$ ksi

$M_{dnc} := M_{nc}$ $\quad\boxed{M_{dnc} = 4887}$ kip-ft

$S_c := -S_{cgb}$ $\quad\boxed{S_c = 24681}$ in^3

$S_{nc} := -S_b$ $\quad\boxed{S_{nc} = 18825}$ in^3

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete structures

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$ $\boxed{M_{cr} = 10551}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and $1.33 \cdot M_{str}$? $\quad\boxed{check = \text{"OK"}}$

The moment capacity looks good, with some over strength for the interior girder. However, we must check the capacity of the exterior girder since the available flange width is less.

Check the exterior girder capacity:

The effective flange width for exterior girder is calculated in accordance with **LRFD [4.6.2.6]** as one half the effective width of the adjacent interior girder plus the overhang width :

$$w_{ex\_oh} := s_{oh} \cdot 12 \qquad \boxed{w_{ex\_oh} = 30.0} \text{ in}$$

$$w_{ex} := \frac{w_e}{2} + w_{ex\_oh} \qquad \boxed{w_{ex} = 75.00} \text{ in}$$

$b_x := w_{ex}$ effective deck width of the compression flange.

Calculate the neutral axis location for a flanged section:

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c_x := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c_x = 13.51} \text{ in}$$

$$a_x := \beta_1 \cdot c_x \qquad \boxed{a_x = 11.49} \text{ in}$$

Now calculate the effective tendon stress at ultimate:

$$f_{ps\_x} := f_{pu} \cdot \left(1 - k \cdot \frac{c_x}{d_p}\right) \qquad \boxed{f_{ps\_x} = 256.759} \text{ ksi}$$

The nominal moment capacity of the composite section (exterior girder) ignoring the increased strength of the concrete in the girder flange:

$$M_{n\_x} := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a_x}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot (b_x - w_{tf}) \cdot h_f \cdot \left(\frac{a_x}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_{n\_x} = 15515} \text{ kip-ft}$$

$$M_{r\_x} := \phi_f \cdot M_{n\_x} \qquad \boxed{M_{r\_x} = 15515} \text{ kip-ft}$$

$\boxed{1.33M_{strx} = 14874}$ kip-ft

Is M_{r_x} greater than $1.33*M_{strx}$? $\boxed{check = "OK"}$

Since M_{r_x} is greater than $1.33*M_{strx}$, the check for M_{cr} does not need to be completed.

E19-1.13 Shear Analysis

A separate analysis must be conducted to estimate the total shear force in each girder for shear design purposes.

Calculate the shear distribution to the girders, **LRFD [Table 4.6.2.2.3a-1]**:

Interior Beams:

One lane loaded:

$$g_{vi1} := 0.36 + \frac{S}{25} \qquad \boxed{g_{vi1} = 0.660}$$

Two or more lanes loaded:

$$g_{vi2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \boxed{g_{vi2} = 0.779}$$

$$g_{vi} := \max\left(g_{vi1}, g_{vi2}\right) \qquad \boxed{g_{vi} = 0.779}$$

Note:The distribution factors above include the multiple lane factor. *The skew correction factor, as now required by a WisDOT policy item for all girders, is omitted. This example is not yet revised.*

Exterior Beams:

Two or more lanes loaded:

The distance from the centerline of the exterior beam to the inside edge of the parapet, $d_e = 1.25$ feet.

$$e_v := 0.6 + \frac{d_e}{10} \qquad \boxed{e_v = 0.725}$$

$$g_{vx2} := e_v \cdot g_{vi} \qquad \boxed{g_{vx2} = 0.565}$$

With a single lane loaded, we use the lever rule (same as before). Note that the multiple presence factor has already been applied to g_{x2}..

$$g_{vx1} := g_{x1} \quad = e \cdot g_i \qquad \boxed{g_{vx1} = 0.600}$$

$g_{vx} := \max(g_{vx1}, g_{vx2})$ $\boxed{g_{vx} = 0.600}$

Apply the shear magnification factor in accordance with **LRFD [4.6.2.2.3c]**.

$$skew_{correction} := 1.0 + 0.2 \cdot \left(\frac{12L \cdot t_s^{\,3}}{K_g}\right)^{0.3} \cdot \tan\left(skew \cdot \frac{\pi}{180}\right)$$

$\boxed{L = 146.00}$

$\boxed{t_s = 8.00}$

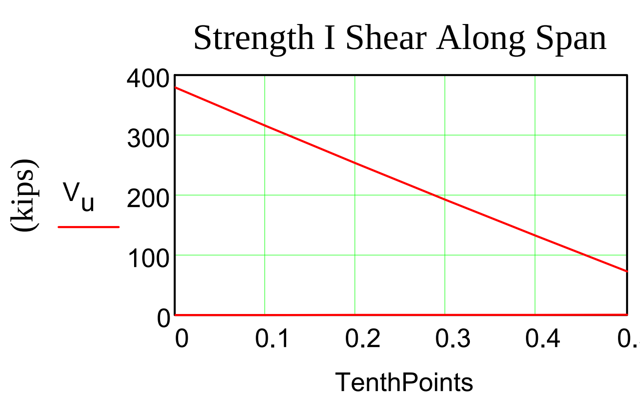
$\boxed{K_g = 3600866}$

$\boxed{skew = 20.000}$ $\boxed{skew_{correction} = 1.048}$

$g_{vx} := g_{vx} \cdot skew_{correction}$ $\boxed{g_{vx} = 0.629}$

The interior girder will control. It has a larger distribution factor and a larger dead load.

Conduct a bridge analysis as before with similar load cases for the maximum girder shear forces. We are interested in the Strength 1 condition now for shear design.



$\boxed{Vu_{0.0} = 379.7}$ kips

$\boxed{Vu_{0.5} = 72.9}$ kips

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := t_w$ $\boxed{b_v = 6.50}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.
d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$$d_v := -e_s + y_t + hau + t_{se} - \frac{a}{2}$$ $\boxed{d_v = 72.50}$ in

However, since there are draped strands for a distance of HD = 49.00 feet from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section. Since the draped strands will raise the center of gravity of the strand group near the girder end, try a smaller value of "d_v" and recalculate "e_s" and "a".

Try $d_v := 65$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right) \cdot \frac{1}{12} + 0.5$$ $\boxed{L_{crit} = 6.25}$ ft

Calculate the eccentricity of the strand group at the critical section.

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100} \cdot L_{crit} \cdot 12$$ $\boxed{y_{8t\_crit} = 24.22}$ in

$$e_{s\_crit} := \frac{ns_s \cdot y_s + ns_d \cdot y_{8t\_crit}}{ns_s + ns_d}$$ $\boxed{e_{s\_crit} = -21.11}$ in

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + hau + t_{se} - e_{s\_crit}$$ $\boxed{d_{p\_crit} = 67.74}$ in

$$A_{ps\_crit} := (ns_d + ns_s) \cdot A_s$$ $\boxed{A_{ps\_crit} = 9.98}$ in^2

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$\boxed{d_b = 0.600}$ in

$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b$ $\boxed{l_d = 146.2}$ in

The transfer length may be taken as: $l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.250$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} + \frac{L_{crit} \cdot 12 - l_{tr}}{l_d - l_{tr}} \cdot \left(f_{ps} - f_{pe}\right)$$ $\boxed{f_{pu\_crit} = 195}$ ksi

For rectangular section behavior:

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \frac{f_{pu\_crit}}{d_{p\_crit}}}$$ $\boxed{c = 7.276}$ in

$a_{crit} := \beta_1 \cdot c$ $\boxed{a_{crit} = 6.184}$ in

Calculation of shear depth based on refined calculations of e_s and a:

$$d_{v\_crit} := -e_{s\_crit} + y_t + hau + t_{se} - \frac{a_{crit}}{2}$$ $\boxed{d_{v\_crit} = 64.65}$ in

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**). Note, the value of V_p does not equal zero in the calculation of V_{cw}.

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kips)

V_i = factored shear force at section due to externally applied loads (Live Loads) occurring simultaneously with M_{max} (kips). (Not necessarily equal to V_u.)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in)

M_{max} = maximum factored moment at section due to externally applied loads (Live Loads) (kip-in)

M_{dnc} = total unfactored dead load moment acting on the noncomposite section (kip-ft)

Values for the following moments and shears are at the critical section, $L_{crit} = 6.25$ feet from the end of the girder at the abutment.

$V_d = 141$ kips

$V_i = 136$ kips

$M_{dnc} = 740$ kip-ft

$M_{max} = 837$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$M_{max} = 10048$ kip-in

$f_r = -0.20 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := -0.20 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = -0.566}$ ksi

$\boxed{T = 1602}$ kips

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_b}$$ $\boxed{f_{cpe} = 3.548}$ ksi

$\boxed{M_{dnc} = 740}$ kip-ft

$\boxed{M_{max} = 10048}$ kip-in

$S_c := S_{cgb}$ $\boxed{S_c = -24681}$ in^3

$S_{nc} := S_b$ $\boxed{S_{nc} = -18825}$ in^3

$$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12 M_{dnc}}{S_{nc}} \right)$$ $\boxed{M_{cre} = 89892}$ kip-in

Calculate V_{ci}, **LRFD [5.8.3.4.3]**   λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{ci1} := 0.06 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 71.7}$ kips

$$V_{ci2} := 0.02 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$$ $\boxed{V_{ci2} = 1384.0}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 1384.0}$ kips

$$f_t := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$$ $\boxed{f_t = 0.340}$ ksi

$$f_b := \frac{T}{A_g} + \frac{T \cdot e_{s\_crit}}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$$ $\boxed{f_b = 3.076}$ ksi

$\boxed{y_{cgb} = -48.76}$ in

$\boxed{ht = 72.00}$ in

$$f_{pc} := f_b - y_{cgb} \cdot \frac{f_t - f_b}{ht}$$ $\boxed{f_{pc} = 1.223}$ ksi

$$V_{p\_cw} := ns_d \cdot A_s \cdot f_{pe} \cdot \frac{slope}{100}$$ $\boxed{V_{p\_cw} = 29.4}$ kips

Calculate V_{cw}, **LRFD [5.8.3.4.3]**   λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{cw} := \left(0.06 \cdot \lambda \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 256.1}$ kips

$V_c := \min\left(V_{ci}, V_{cw}\right)$ $\boxed{V_c = 256.1}$ kips

Calculate the required shear resistance:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$V_{u\_crit} = \gamma st_{DC} \cdot \left(V_{DCnc} + V_{DCc}\right) + \gamma st_{DW} \cdot V_{DWc} + \gamma st_{LL} \cdot Vu_{LL}$ where,

$V_{DCnc} = 123.357$ kips $V_{DCc} = 8.675$ kips $V_{DWc} = 8.967$ kips $Vu_{LL} = 100.502$ kips

$V_{u\_crit} = 354.368$ kips $V_n := \dfrac{V_{u\_crit}}{\phi_v}$ $\boxed{V_n = 393.7}$ kips

Transverse Reinforcing Design at Critical Section:

The required steel capacity:

$V_s := V_n - V_c - V_p$ $\boxed{V_s = 137.7}$ kips

$A_v := 0.40$ in^2 for #4 rebar

$f_y := 60$ ksi

$\boxed{d_v = 65.00}$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3 \cdot \dfrac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$ $\boxed{\cot\theta = 1.800}$

$V_s = A_v \cdot f_y \cdot d_v \cdot \dfrac{\cot\theta}{s}$

LRFD Eq 5.8.3.3-4 reduced per **C5.8.3.3-1** when α = 90 degrees.

$s := A_v \cdot f_y \cdot d_v \cdot \dfrac{\cot\theta}{V_s}$ $\boxed{s = 20.399}$ in

Check Maximum Spacing, **LRFD [5.8.2.7]**:

$v_u := \dfrac{V_{u\_crit}}{\phi_v \cdot b_v \cdot d_v}$ $\boxed{v_u = 0.932}$ ksi

Max. stirrup spacing per WisDOT policy item is 18" $\boxed{0.125 \cdot f'_c = 1.000}$ ksi

$$s_{max1} := \begin{cases} \min(0.8 \cdot d_v, 18) & \text{if } v_u < 0.125 \cdot f'_c \\ \min(0.4 \cdot d_v, 12) & \text{if } v_u \geq 0.125 \cdot f'_c \end{cases}$$

$\boxed{s_{max1} = 18.00}$ in

Check Minimum Reinforcing, **LRFD [5.8.2.5]**:

$$s_{max2} := \frac{A_v \cdot f_y}{0.0316 \cdot \lambda \sqrt{f'_c} \cdot b_v}$$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$\boxed{s_{max2} = 41.31}$ in

$$s_{max} := \min(s_{max1}, s_{max2})$$

$\boxed{s_{max} = 18.00}$ in

Therefore use a maximum spacing of $s := 18$ inches.

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$

$\boxed{V_s = 156}$ kips

Check V_n requirements:

$$Vn_1 := V_c + V_s + V_p$$

$\boxed{Vn_1 = 412}$ kips

$$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$$

$\boxed{Vn_2 = 845}$ kips

$$V_n := \min(Vn_1, Vn_2)$$

$\boxed{V_n = 412}$ kips

$$V_r := \phi_v \cdot V_n$$

$\boxed{V_r = 370.88}$ kips

$\boxed{V_{u\_crit} = 354.37}$ kips

Is V_{u_crit} less than V_r?

$\boxed{check = \text{"OK"}}$

Web reinforcing is required in accordance with **LRFD [5.8.2.4]** whenever:

$$V_u \geq 0.5 \cdot \phi_v \cdot (V_c + V_p)$$

(all values shown are in kips)

At critical section from end of girder: $\boxed{V_{u\_crit} = 354}$ $\boxed{0.5 \cdot \phi_v \cdot (V_c + V_p) = 115}$

From calculations similar to those shown above,

At hold down point: $\boxed{V_{u\_hd} = 172}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_hd} + V_p) = 64}$

At mid-span: $\boxed{V_{u\_mid} = 73}$ $\boxed{0.5 \cdot \phi_v \cdot (V_{c\_mid} + V_p) = 36}$

Therefore, use web reinforcing over the entire beam.

Resulting Shear Design:

Use #4 U shaped stirrups at 18-inch spacing between the typical end sections. Unless a large savings in rebar can be realized, use a single stirrup spacing between the standard end sections.

E19-1.14 Longitudinal Tension Flange Capacity:

The total capacity of the tension reinforcing must meet the requirements of **LRFD [5.8.3.5].** The capacity is checked at the critical section for shear:

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw} \right) \cdot \cot\theta \qquad \boxed{T_{ps} = 670} \text{ kips}$$

actual capacity of the straight strands:

$$\boxed{ns_s \cdot A_s \cdot f_{pu\_crit} = 1612} \text{ kips}$$

Is the capacity of the straight strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

Check the tension Capacity at the edge of the bearing:

The strand is anchored $l_{px} := 10$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.11.4.2]**:

$$\boxed{l_{tr} = 36.00} \text{ in}$$

$$\boxed{l_d = 146.2} \text{ in}$$

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$$l_{px'} := l_{px} + Y_s \cdot \cot\theta \qquad \boxed{Y_s = 4.21} \text{ in} \qquad \boxed{l_{px'} = 17.58} \text{ in}$$

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_b} \qquad \boxed{f_{pb} = 78.37} \text{ ksi}$$

Tendon capacity of the straight strands: $\boxed{ns_s \cdot A_s \cdot f_{pb} = 646}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$$s_{ave} := \frac{6 \cdot 4.25 + 6 \cdot 5.5}{12}$$ $\boxed{s_{ave} = 4.88}$ in

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s_{ave}}$$ $\boxed{V_s = 576}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 29}$ kips

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 354}$ kips

Minimum capacity required at the front of the bearing:

$$T_{breqd} := \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw} \right) \cdot \cot\theta$$ $\boxed{T_{breqd} = 137}$ kips

Is the capacity of the straight strands greater than T_{breqd}? $\boxed{check = \text{"OK"}}$

E19-1.15 Composite Action - Design for Interface Shear Transfer

The total shear to be transferred to the flange between the end of the beam and mid-span is equal to the compression force in the compression block of the flange and haunch in strength condition. For slab on girder bridges, the shear interface force is calculated in accordance with **LRFD [5.8.4.2]**.

$b_{vi} := 18$ in width of top flange available to bond to the deck

$\boxed{d_v = 65.00}$ in

$$v_{ui} := \frac{V_{u\_crit}}{b_{vi} \cdot d_v}$$ $\boxed{v_{ui} = 0.303}$ ksi

$$V_{ui} := v_{ui} \cdot 12 \cdot b_{vi}$$ $\boxed{V_{ui} = 65.4}$ kips/ft

$$V_n = c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right)$$ **LRFD [5.8.4.1]**

The nominal shear resistance, V_n, used in design shall not be greater than the lesser of:

$V_{n1} = K_1 \cdot f'_{cd} \cdot A_{cv}$ or $V_{n2} = K_2 \cdot A_{cv}$

$c := 0.28$ ksi

$\mu := 1.0$

$K_1 := 0.3$

$K_2 := 1.8$

$A_{cv} := b_{vi} \cdot 12$ Area of concrete considered to be engaged in interface shear transfer. $\boxed{A_{cv} = 216}$ in²/ft

For an exterior girder, P_c is the weight of the deck, haunch, parapet and FWS.

$$P_{cd} := \frac{w_c \frac{t_s}{12}}{2 \cdot S} \cdot (S + s_{oh})^2 \qquad \boxed{P_{cd} = 0.667} \text{ klf}$$

$$P_{ch} := \frac{hau \cdot w_{tf}}{12^2} \cdot w_c \qquad \boxed{P_{ch} = 0.100} \text{ klf}$$

$$P_{cp} := w_p \qquad \boxed{P_{cp} = 0.129} \text{ klf}$$

$$P_{cfws} := w_{ws} \qquad \boxed{P_{cfws} = 0.133} \text{ klf}$$

$$P_c := P_{cd} + P_{ch} + P_{cp} + P_{cfws} \qquad \boxed{P_c = 1.029} \text{ klf}$$

From earlier calculations, the maximum #4 stirrup spacing used is $s = 18.0$ inches.

$$A_{vf} := \frac{A_v}{s} \cdot 12 \qquad \boxed{A_{vf} = 0.267} \text{ in}^2\text{/ft}$$

$$V_n := c \cdot A_{cv} + \mu \cdot (A_{vf} \cdot f_y + P_c) \qquad \boxed{V_n = 77.5} \text{ kips/ft}$$

$$V_{n1} := K_1 \cdot f'_{cd} \cdot A_{cv} \qquad \boxed{V_{n1} = 259.2} \text{ kips/ft}$$

$$V_{n2} := K_2 \cdot A_{cv} \qquad \boxed{V_{n2} = 388.8} \text{ kips/ft}$$

$$V_n := \min(V_n, V_{n1}, V_{n2}) \qquad \boxed{V_n = 77.5} \text{ kips/ft}$$

$$V_r := \phi_v \cdot V_n \qquad \boxed{V_r = 69.8} \text{ kips/ft}$$

$$\boxed{V_{ui} = 65.4} \text{ kips/ft}$$

Is V_r greater than V_{ui}? $\boxed{check = \text{"OK"}}$

Solution:
#4 stirrups spaced at $s = 18.0$ inches is adequate to develop the required interface shear

resistance for the entire length of the girder.

E19-1.16 Deflection Calculations

Check the Live Load deflection with the vehicle loading as specified in **LRFD [3.6.1.3.2]**; design truck alone or 25% of the design truck + the lane load.

The deflection shall be limited to L/800.

The moment of inertia of the entire bridge shall be used.

$$\Delta_{limit} := \frac{L \cdot 12}{800}$$ $\boxed{\Delta_{limit} = 2.190}$ inches

$I_{cg} = 1203475.476$

$ng = 6$ number of girders

$$I_{bridge} := I_{cg} \cdot ng$$ $\boxed{I_{bridge} = 7220853}$ in^4

From CBA analysis with 3 lanes loaded, the truck deflection controlled:

$\Delta_{truck} := 0.648$ in

Applying the multiple presence factor from **LRFD Table [3.6.1.1.2-1]** for 3 lanes loaded:

$$\Delta := 0.85 \cdot \Delta_{truck}$$ $\boxed{\Delta = 0.551}$ in

Is the actual deflection less than the allowable limit, $\Delta < \Delta$ limit? $\boxed{check = "OK"}$

E19-1.17 Camber Calculations

Moment due to straight strands:

Number of straight strands: $\boxed{ns_s = 38}$

Eccentricity of the straight strands: $y_s = -30.66$ in

$P_{i\_s} := ns_s \cdot A_s \cdot (f_{tr} - \Delta f_{pES})$ $P_{i\_s} = 1520$ kips

$M_1 := P_{i\_s} \cdot |y_s|$ $M_1 = 46598$ kip-in

Upward deflection due to straight strands:

Length of the girder: $L_g = 147$ ft

Modulus of Elasticity of the girder at release: $E_{ct} = 4999$ ksi

Moment of inertia of the girder: $I_g = 656426$ in^4

$$\Delta_s := \frac{M_1 \cdot L_g^2}{8 \cdot E_{ct} \cdot I_g} \cdot 12^2$$ $\Delta_s = 5.523$ in

Moment due to draped strands:

$P_{i\_d} := ns_d \cdot A_s \cdot (f_{tr} - \Delta f_{pES})$ $P_{i\_d} = 319.971$ kips

$A = 67.000$ in

$C = 5.000$ in

$M_2 := P_{i\_d} \cdot (A - C)$ $M_2 = 19838.175$ kip-in

$M_3 := P_{i\_d} \cdot (A - |y_b|)$ $M_3 = 10280.654$ kip-in

Upward deflection due to draped strands:

$$\Delta_d := \frac{L_g^2}{8 \cdot E_{ct} \cdot I_g} \cdot \left(\frac{23}{27} \cdot M_2 - M_3\right) \cdot 12^2$$ $\Delta_d = 0.784$ in

Total upward deflection due to prestress:

$\Delta_{PS} := \Delta_s + \Delta_d$ $\Delta_{PS} = 6.308$ in

Downward deflection due to beam self weight at release:

$$\Delta_{gi} := \frac{5 \cdot w_g \cdot L^4}{384 \cdot E_{ct} \cdot I_g} \cdot 12^3$$ $\Delta_{gi} = 2.969$ in

Anticipated prestress camber at release:

$\Delta_i := \Delta_{PS} - \Delta_{gi}$ $\boxed{\Delta_i = 3.339}$ in

The downward deflection due to the dead load of the deck and diaphragms:

Calculate the additional non-composite dead loads for an interior girder:

$w_{nc} := w_{dlii} - w_g$ $\boxed{w_{nc} = 0.881}$ klf

Modulus of Elasticity of the beam at final strength $\boxed{E_B = 6351}$ ksi

$$\Delta_{nc} := \frac{5 \cdot w_{nc} \cdot L^4}{384 \cdot E_B \cdot I_g} \cdot 12^3$$ $\boxed{\Delta_{nc} = 2.161}$ in

The downward deflection due to the dead load of the parapets is calculated as follows. Note that the deflections due to future wearing surface loads are not considered.

Calculate the composite dead loads for an interior girder:

$w_{ws} := 0$ klf

$w_c := w_p + w_{ws}$ $\boxed{w_c = 0.129}$ klf

$$\Delta_c := \frac{5 \cdot w_c \cdot L^4}{384 \cdot E_B \cdot I_{cg}} \cdot 12^3$$ $\boxed{\Delta_c = 0.173}$ in

The total downward deflection due to dead loads acting on an interior girder:

$\Delta_{DL} := \Delta_{nc} + \Delta_c$ $\boxed{\Delta_{DL} = 2.334}$ in

The residual camber for an interior girder:

$RC := \Delta_i - \Delta_{DL}$ $\boxed{RC = 1.005}$ in

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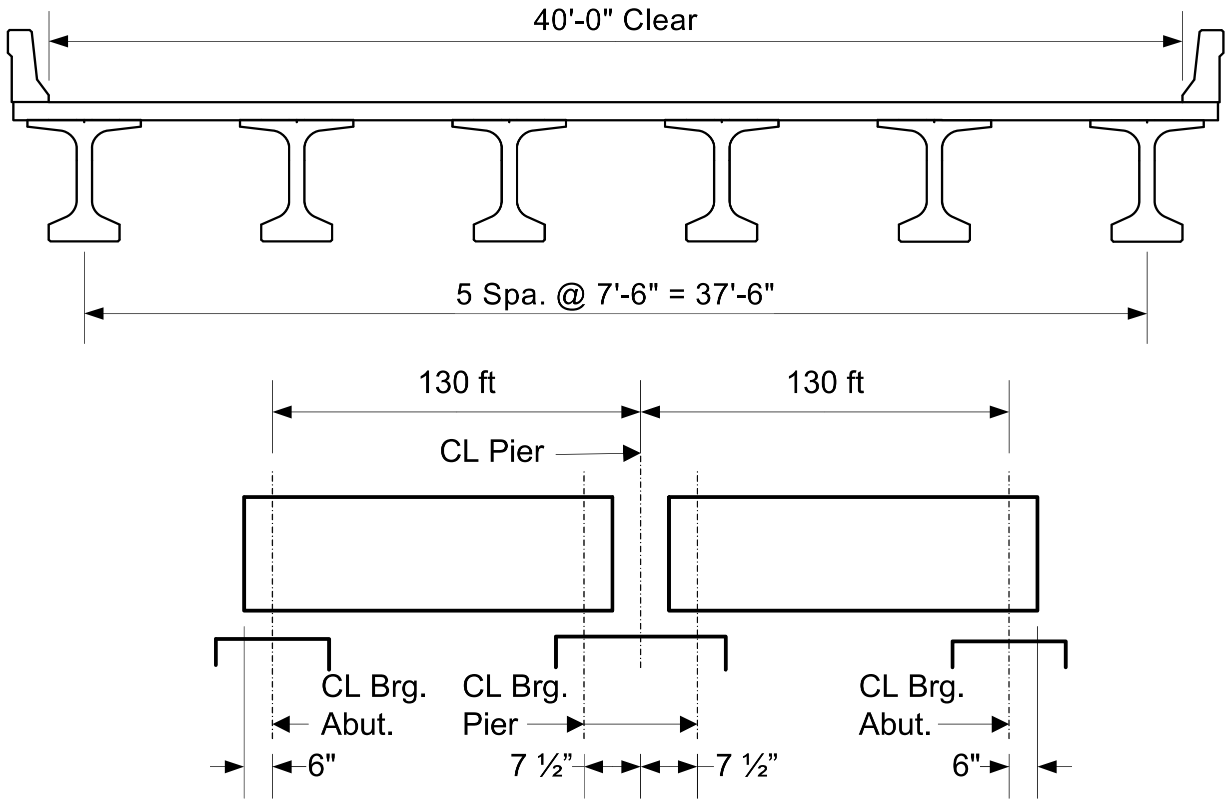
E19-2 Two-Span 54W" Girder, Continuity Reinforcement - LRFD

This example shows design calculations for the continuity reinforcement for a two span prestressed girder bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter.***(Example is current through LRFD Seventh Ed. - 2016 Int.)***

E19-2.1 Design Criteria



$L := 130$	center of bearing at abutment to CL pier for each span, ft
$L_g := 130.375$	total length of the girder (the girder extends 6 inches past the center of bearing at the abutment and 1.5" short of the center line of the pier).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_y := 60$	yield strength of mild reinforcement, ksi

$w_p := 0.387$ weight of Wisconsin Type LF parapet, klf

$t_s := 8$ slab thickness, in

$t_{se} := 7.5$ effective slab thickness, in

$skew := 0$ skew angle, degrees

$w_c := 0.150$ kcf

$E_s := 29000$ ksi, Modulus of Elasticity of the reinforcing steel

E19-2.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351} \qquad E_B := E_{beam8}$$

$$E_D := E_{deck4}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

E19-2.3 Section Properties

54W Girder Properties:

$w_{tf} := 48$ in

$t_t := 4.625$ in

$t_w := 6.5$ in

$t_b := 10.81$ in

$ht := 54$ in

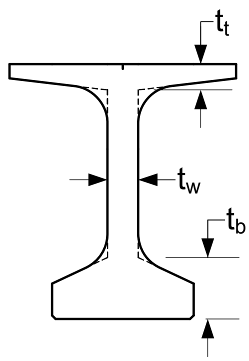
$b_w := 30$ width of bottom flange, in

$A_g := 798$ in^2

$I_g := 321049$ in^4

$y_t := 27.70$ in

$y_b := -26.30$ in



$$e_g := y_t + 2 + \frac{t_{se}}{2} \qquad \boxed{e_g = 33.45} \text{ in}$$

$S_t := 11592$ in^3

$S_b := -12205$ in^3

LRFD [Eq 4.6.2.2.1-1]

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ $\boxed{K_g = 1868972}$ in^4

E19-2.4 Girder Layout

Chapter 19 suggests that at a 130 foot span, the girder spacing should be 7'-6" with 54W girders.

$S := 7.5$ ft

Assume a minimum overhang of 2.5 feet (2 ft flange + 6" overhang), $s_{oh} := 2.5$

$ns := \dfrac{w_b - s_{oh}}{S}$ $\boxed{ns = 5.333}$

Use the lowest integer: $ns := \text{floor}(ns)$ $\boxed{ns = 5}$

Number of girders: $ng := ns + 1$ $\boxed{ng = 6}$

Overhang Length: $s_{oh} := \dfrac{w_b - S \cdot ns}{2}$ $\boxed{s_{oh} = 2.50}$ ft

E19-2.5 Loads

$w_g := 0.831$	weight of 54W girders, klf
$w_d := 0.100$	weight of 8-inch deck slab (interior), ksf
$w_h := 0.100$	weight of 2-in haunch, klf
$w_{di} := 0.410$	weight of diaphragms on interior girder (assume 2), kips
$w_{dx} := 0.205$	weight of diaphragms on exterior girder, kips
$w_{ws} := 0.020$	future wearing surface, ksf
$w_p = 0.387$	weight of parapet, klf

E19-2.5.1 Dead Loads

Dead load on non-composite (DC):

exterior:

$w_{dlxi} := w_g + w_d \cdot \left(\dfrac{S}{2} + s_{oh}\right) + w_h + 2 \cdot \dfrac{w_{dx}}{L}$ $\boxed{w_{dlxi} = 1.559}$ klf interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L}$$ $\boxed{w_{dlii} = 1.687}$ klf

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng}$$ $\boxed{w_p = 0.129}$ klf

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng}$$ $\boxed{w_{ws} = 0.133}$ klf

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E19-2.5.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = truck + lane **LRFD [3.6.1.3.1]**
truck pair + lane

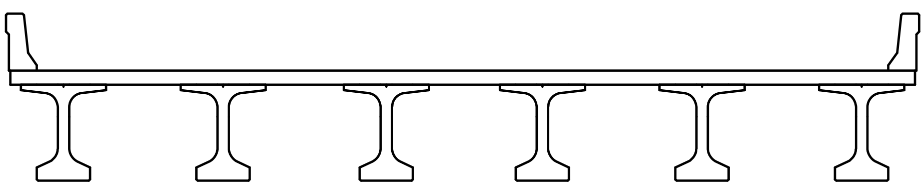
DLA of 33% applied to truck or tandem, but not to lane per **LRFD [3.6.2.1].**

For Fatigue 1:

HL-93 truck (no lane) with 15% DLA and 30 ft rear axle spacing per **LRFD [3.6.1.4.1].**

E19-2.6 Load Distribution to Girders

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1}$$

Criteria for using distribution factors - Range of Applicability per **LRFD [Table 4.6.2.2.2b-1]**.

$$DeckSpan := \left| \begin{array}{ll} \text{"OK"} & \text{if } 3.5 \le S \le 16 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$DeckThickness := \left| \begin{array}{ll} \text{"OK"} & \text{if } 4.5 \le t_{se} \le 12 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$BridgeSpan := \left| \begin{array}{ll} \text{"OK"} & \text{if } 20 \le L \le 240 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$NoBeams := \left| \begin{array}{ll} \text{"OK"} & \text{if } ng \ge 4 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$LongitStiffness := \left| \begin{array}{ll} \text{"OK"} & \text{if } 10000 \le K_g \le 7000000 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$x := \begin{pmatrix} S & DeckSpan \\ t_{se} & DeckThickness \\ L & BridgeSpan \\ ng & NoBeams \\ K_g & LongitStiffness \end{pmatrix} \qquad x = \begin{pmatrix} 7.5 & \text{"OK"} \\ 7.5 & \text{"OK"} \\ 130.0 & \text{"OK"} \\ 6.0 & \text{"OK"} \\ 1868972.4 & \text{"OK"} \end{pmatrix}$$

E19-2.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1} \qquad \boxed{g_{i1} = 0.427}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$ $\boxed{g_{i2} = 0.619}$

$$g_i := \max\left(g_{i1}, g_{i2}\right)$$ $\boxed{g_i = 0.619}$

Note: The distribution factors above already have a multiple lane factor included that is used for service and strength limit states. The distribution factor for One Lane Loaded should be used for the fatigue vehicle and the 1.2 multiple presence factor should be divided out.

E19-2.6.2 Distribution Factors for Exterior Beams:

Two or More Lanes Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the following equations:

$$w_{parapet} := \frac{w_b - w}{2}$$ Width of parapet overlapping the deck $\boxed{w_{parapet} = 1.250}$ ft

$$d_e := s_{oh} - w_{parapet}$$ Distance from the exterior web of exterior beam to the interior edge of parapet, ft. $\boxed{d_e = 1.250}$ ft

Note: Conservatively taken as the distance from the center of the exterior girder.

Check range of applicability for de:

$$d_{e\_check} := \begin{cases} \text{"OK"} & \text{if } -1.0 \le d_e \le 5.5 \\ \text{"NG"} & \text{otherwise} \end{cases}$$ $\boxed{d_{e\_check} = \text{"OK"}}$

Note: While AASHTO allows the d_e value to be up to 5.5, the deck overhang (from the center of the exterior girder to the edge of the deck) is limited by WisDOT policy as stated in Chapter 17 of the Bridge Manual.

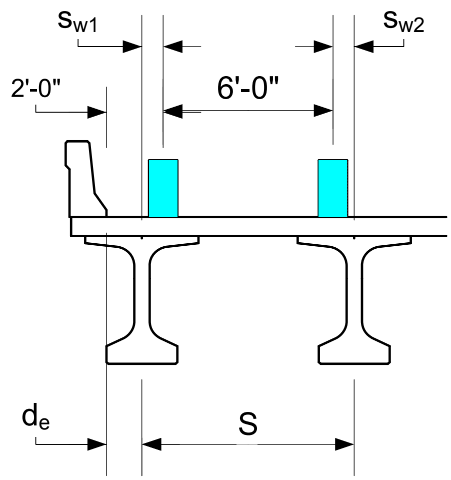
$$e := 0.77 + \frac{d_e}{9.1}$$ $\boxed{e = 0.907}$

$$g_{x1} := e \cdot g_i$$ $\boxed{g_{x1} = 0.562}$

One Lane Loaded:

Per **LRFD [Table 4.6.2.2.2d-1]** the distribution factor shall be calculated by the Lever Rule.

Calculate the distribution factor by the Lever Rule:



$s_{w1} := d_e - 2$ Distance from center of exterior girder to outside wheel load, ft. $\boxed{s_{w1} = -0.75}$ ft

$s_{w2} := S + s_{w1} - 6$ Distance from wheel load to first interior girder, ft. $\boxed{s_{w2} = 0.75}$ ft

$R_x := \frac{S + s_{w1} + s_{w2}}{S \cdot 2}$ $\boxed{R_x = 0.500}$ % of a lane load

Add the single lane multi-presence factor, $m := 1.2$

$g_{x2} := R_x \cdot 1.2$ $\boxed{g_{x2} = 0.600}$

The exterior girder distribution factor is the maximum value of the One Lane Loaded case and the Two or More Lanes Loaded case:

$g_x := \max(g_{x1}, g_{x2})$ $\boxed{g_x = 0.600}$

Note: The interior girder has a larger live load distribution factor and a larger dead load than the exterior girder. Therefore, for this example, the interior girder is likely to control.

E19-2.7 Load Factors

From **LRFD [Table 3.4.1-1]**:

	DC	DW	LL
Strength 1	$\gamma st_{DC} := 1.25$	$\gamma st_{DW} := 1.50$	$\gamma st_{LL} := 1.75$
Service 1	$\gamma s1_{DC} := 1.0$	$\gamma s1_{DW} := 1.0$	$\gamma s1_{LL} := 1.0$
Fatigue 1	$\gamma f_{DC} := 1.0$	$\gamma f_{DW} := 1.0$	$\gamma f_{LL} := 1.50$

Impact factor (DLA) is applied to the truck and tandem.

E19-2.8 Dead Load Moments

The unfactored dead load moments are listed below (values are in kip-ft):

Unfactored Dead Load Interior Girder Moments (ft-kips)			
Tenth Point	DC non-composite	DC composite	DW composite
0.5	3548	137	141
0.6	3402	99	102
0.7	2970	39	40
0.8	2254	-43	-45
0.9	1253	-147	-151
1.0	0	-272	-281

The DC_{nc} values are the component non-composite dead loads and include the weight of the girder, haunch, diaphragms and the deck.

The DC_c values are the component composite dead loads and include the weight of the parapets.

The DW_c values are the composite dead loads from the future wearing surface.

Note that the girder dead load moments (a portion of DC_{nc}) are calculated based on the CL bearing to CL bearing length. The other DC_{nc} moments are calculated based on the span length (center of bearing at the abutment to centerline of the pier).

E19-2.9 Live Load Moments

The unfactored live load moments (per lane including impact) are listed below (values are in kip-ft). Note that the impact factor is applied only to the truck portion of the HL-93 loads. A separate analysis run will be required if results without impact are desired.

Unfactored Live Load + Impact Moments per Lane (kip-ft)

Tenth Point	Truck Pair	Truck + Lane	- Fatigue	+ Fatigue
0.5	--	-921	-476	1644
0.6	--	-1106	-572	1497
0.7	--	-1290	-667	1175
0.8	-1524	-1474	-762	718
0.9	-2046	-1845	-857	262
1	-3318	-2517	-953	0

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$g_i = 0.619$

$M_{LL} = g_i \cdot -3317.97$ $\boxed{M_{LL} = -2055}$ kip-ft

The single lane distribution factor should be used and the multiple presence factor of 1.2 must be removed from the fatigue moments.

$M_{LLfatigue} = g_{i1} \cdot -952.64 \cdot \frac{1}{1.2}$ $\boxed{M_{LLfatigue} = -339}$ kip-ft

E19-2.10 Factored Moments

The factored moments for each limit state are calculated by applying the appropriate load factor to the girder moments. For the interior girder:

Strength 1

$M_u := \eta \cdot (\gamma st_{DC} \cdot M_{DCc} + \gamma st_{DW} \cdot M_{DWc} + \gamma st_{LL} \cdot M_{LL})$

$= 1.0 \cdot (1.25 \cdot M_{DCc} + 1.50 \cdot M_{DWc} + 1.75 \cdot M_{LL})$ $\boxed{M_u = -4358}$ kip-ft

Service 1 (for compression checks in prestress and crack control in deck)

$M_{s1} := \eta \cdot (\gamma s1_{DC} \cdot M_{DCc} + \gamma s1_{DW} \cdot M_{DWc} + \gamma s1_{LL} \cdot M_{LL})$

$= 1.0 \cdot (1.0 \cdot M_{DCc} + 1.0 \cdot M_{DWc} + 1.0 \cdot M_{LL})$ $\boxed{M_{s1} = -2608}$ kip-ft

Fatigue 1

$M_f := \eta \cdot (\gamma f_{DC} \cdot M_{DCc} + \gamma f_{DW} \cdot M_{DWc} + \gamma f_{LL} \cdot M_{LLfatigue})$

$= 1.0 \cdot (1.0 \cdot M_{DCc} + 1.0 \cdot M_{DWc} + 1.50 \cdot M_{LLfatigue})$ $\boxed{M_f = -1062}$ kip-ft

$Mf_{DL} := \eta \cdot (\gamma f_{DC} \cdot M_{DCc} + \gamma f_{DW} \cdot M_{DWc})$ $\boxed{Mf_{DL} = -553}$ kip-ft

$Mf_{LL} := \eta \cdot \gamma f_{LL} \cdot M_{LLfatigue}$ $\boxed{Mf_{LL} = -509}$ kip-ft

E19-2.11 Composite Girder Section Properties

Calculate the effective flange width in accordance with Chapter 17.2.11.

$w_e := S \cdot 12$ $\boxed{w_e = 90.00}$ in

The effective width, w_e, must be adjusted by the modular ratio, $n = 1.54$, to convert to the same concrete material (modulus) as the girder.

$w_{eadj} := \frac{w_e}{n}$ $\boxed{w_{eadj} = 58.46}$ in

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

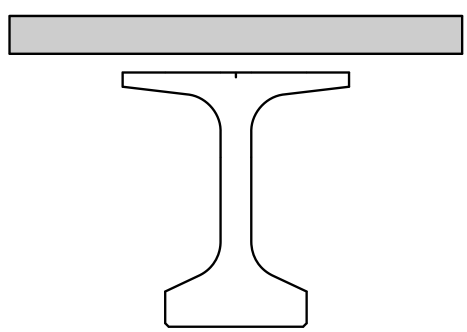
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

haunch thickness; $hau := 2.00$ in

total height; $h_c := ht + hau + t_{se}$

$\boxed{h_c = 63.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	$I+AY^2$
Deck	59.75	438	26197	1565294	2055	1567349
Girder	26.3	798	20987	551969	321049	873018
Haunch	55	0	0	0	0	0
Summation		1236	47185			2440367

$\Sigma A := 1236$ in^2

$\Sigma AY := 47185$ in^4

$\Sigma IplusAYsq := 2440367$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A} \qquad \boxed{y_{cgb} = -38.2} \text{ in}$$

$$y_{cgt} := ht + y_{cgb} \qquad \boxed{y_{cgt} = 15.8} \text{ in}$$

$A_{cg} := \Sigma A$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^2 \qquad \boxed{I_{cg} = 639053} \text{ in}^4$$

Deck:

$$S_c := n \cdot \frac{I_{cg}}{y_{cgt} + hau + t_{se}} \qquad \boxed{S_c = 38851} \text{ in}^4$$

E19-2.12 Flexural Strength Capacity at Pier

All of the continuity reinforcement shall be placed in the top mat. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

$bar_{trans} := 5$ (transverse bar size)

$Bar_D(bar_{trans}) = 0.625$ in (transverse bar diameter)

$Bar_{No} = 9$

$Bar_D(Bar_{No}) = 1.13$ in (Assumed bar size)

$$d_e := ht + hau + t_s - cover - Bar_D(bar_{trans}) - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_e = 60.31} \text{ in}$$

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

$$R_u := \frac{M_u \cdot 12}{\phi_f \cdot b_w \cdot d_e^{\,2}} \qquad \boxed{R_u = 0.532} \text{ ksi}$$

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right) \qquad \boxed{\rho = 0.00925}$$

$$A_s := \rho \cdot b_w \cdot d_e \qquad \boxed{A_s = 16.74} \text{ in}^2$$

This reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches. The required continuity reinforcement in in^2/ft is equal to:

$$As_{req} := \frac{A_s}{\frac{w_e}{12}} \qquad \boxed{As_{req} = 2.232} \text{ in}^2\text{/ft}$$

From Chapter 17, Table 17.5-3, for a girder spacing of $S = 7.5$ feet and a deck thickness of $t_s = 8.0$ inches, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement shall be placed at 1/2 of this bar spacing, .

#9 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 2.82}$ in^2/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12} \qquad \boxed{As = 21.18} \text{ in}^2$$

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

Assume $f_s = f_y$ **LRFD [5.7.2.2]** $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi)

$$a := \frac{As \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c} \qquad \boxed{a = 6.228} \text{ in}$$

This is within the thickness of the bottom flange height of 7.5 inches.

If $\frac{c}{d_s} \le 0.6$ for ($f_y = 60$ ksi) **LRFD [5.7.2.1]**, the reinforcement has yielded and the assumption is correct.

LRFD [5.7.2.2] $\beta_1 := 0.65$; $c := \frac{a}{\beta_1}$ $\boxed{c = 9.582}$ in

$\frac{c}{d_s} = 0.16 < 0.6$ therefore, the reinforcement will yield

$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12}$ $\boxed{M_n = 6056}$ kip-ft

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 5451}$ kip-ft

$\boxed{M_u = 4358}$ kip-ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24 \cdot \lambda \sqrt{f'_{cd}}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_{cd}}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.480}$ ksi

$M_{cr} = \gamma_3 (\gamma_1 \cdot f_r) S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 1709}$ kip-ft

$\boxed{1.33 \cdot M_u = 5796}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho := \frac{As}{b_w \cdot d_e}$ $\boxed{\rho = 0.01170}$

$n := \frac{E_s}{E_B}$ $\boxed{n = 4.566}$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$ $\boxed{k = 0.278}$

$j := 1 - \frac{k}{3}$ $\boxed{j = 0.907}$

Note that the value of d_c should not include the 1/2-inch wearing surface.

$$d_c := cover - 0.5 + Bar_D(bar_{trans}) + \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_c = 3.19}$ in

$\boxed{M_{s1} = 2608}$ kip-ft

$$f_s := \frac{M_{s1}}{As \cdot j \cdot d_e} \cdot 12 \le 0.6\, f_y$$ $\boxed{f_s = 27.006}$ ksi $\le 0.6\, f_y$ O.K.

The height of the composite section, h, is:

$$h := ht + hau + t_{se}$$ $\boxed{h = 63.500}$ in

$$\beta := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$ $\boxed{\beta = 1.076}$

$\gamma_e := 0.75$ for Class 2 exposure

$$S_{max} := \frac{700 \gamma_e}{\beta \cdot f_s} - 2 \cdot d_c$$ $\boxed{S_{max} = 11.70}$ in

$\boxed{spa = 4.25}$ in

Is the bar spacing less than S_{max}? $\boxed{check = \text{"OK"}}$

Check the Fatigue 1 reinforcement limits in accordance with **LRFD [5.5.3]**:

$$\gamma f_{LL} \cdot \Delta f \le \Delta F_{TH} \quad \text{where} \quad \Delta F_{TH} := 24 - 20 \frac{f_{min}}{f_y}$$

$$\Delta F_{TH} := 24 - 0.33\, f_{min} \quad \text{(for } f_y = 60 \text{ ksi)}$$

f_{min} is equal to the stress in the reinforcement due to the moments from the permanent loads combined with the Fatigue I load combination. Δf is the stress range resulting from the fatigue vehicle.

Check stress in section for determination of use of cracked or uncracked section properties:

$$f_{top} := \frac{M_f}{S_c} \cdot 12$$ $\boxed{f_{top} = 0.328}$ ksi

$$f_{limit} := 0.095 \cdot \sqrt{f'_c}$$ $\boxed{f_{limit} = 0.269}$ ksi

Therefore: $\boxed{SectionProp = \text{"Cracked"}}$

If we assume the neutral axis is in the bottom flange, the distance from cracked section neutral axis to bottom of compression flange, y_{cr}, is calculated as follows:

$$\frac{b_w \cdot y_{cr}^2}{2} = n \cdot As \cdot (d_e - y_{cr})$$

$$y_{cr} := \frac{n \cdot As}{b_w} \cdot \left(\sqrt{1 + \frac{2 \cdot b_w \cdot d_e}{n \cdot As}} - 1 \right)$$ $\boxed{y_{cr} = 16.756}$ in No Good

Assume the neutral axis is in the web:

$t_{bf\_min} := 7.5$

$t_{bf\_max} := 15$ $\quad t_{taper} := t_{bf\_max} - t_{bf\_min}$ $\quad \boxed{t_{taper} = 7.500}$

$t_{web} := 7$ $\quad w_{taper} := b_w - t_w$ $\quad \boxed{w_{taper} = 23.500}$

$$(w_{taper}) \cdot t_{bf\_min} \cdot \left(x - \frac{t_{bf\_min}}{2} \right) + t_w \cdot \frac{x^2}{2} \ldots + \left(\frac{w_{taper} \cdot t_{taper}}{2} \right) \cdot \left(x - t_{bf\_min} - \frac{t_{taper}}{3} \right) - n \cdot As \cdot (d_e - x) = 0$$

CG of cracked section, $\boxed{x = 17.626}$ in

Cracked section moment of inertia:

$$I_{cr} := \frac{w_{taper} \cdot t_{bf\_min}^3}{12} + w_{taper} \cdot t_{bf\_min} \cdot \left(x - \frac{t_{bf\_min}}{2} \right)^2 + \frac{t_{web} \cdot x^3}{3} \ldots + \frac{w_{taper} \cdot t_{taper}^3}{36} + \frac{w_{taper} \cdot t_{taper}}{2} \cdot \left(x - t_{bf\_min} - \frac{t_{taper}}{2} \right)^2 + n \cdot As \cdot (d_e - x)^2$$

$\boxed{I_{cr} = 227583}$ in^4

Distance from centroid of tension reinforcement to the cracked section neutral axis:

$y_{rb} := d_e - x$ $\quad \boxed{y_{rb} = 42.685}$ in

$$f_{min} := n \cdot \frac{M_f \cdot y_{rb}}{I_{cr}} \cdot 12$$ $\boxed{f_{min} = 10.913}$ ksi

$\Delta F_{TH} := 24 - 0.33 \cdot f_{min}$ (for f_y = 60 ksi) $\quad \boxed{\Delta F_{TH} = 20.399}$ ksi

$\Delta f := n \cdot \frac{|M_{LLfatigue}| \cdot y_{rb}}{I_{cr}} \cdot 12$ $\boxed{\Delta f = 3.488}$ ksi

$\boxed{\gamma f_{LL} \cdot \Delta f = 5.232}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{check = \text{"OK"}}$

E19-2.13 Bar Cut Offs

The first cut off is located where half of the continuity reinforcement satisfies the moment diagram. Non-composite moments from the girder and the deck are considered along with the composite moments when determining the Strength I moment envelope. (It should be noted that since the non-composite moments are opposite in sign from the composite moments in the negative moment region, the minimum load factor shall be applied to the non-composite moments.) Only the composite moments are considered when checking the Service and Fatigue requirements.

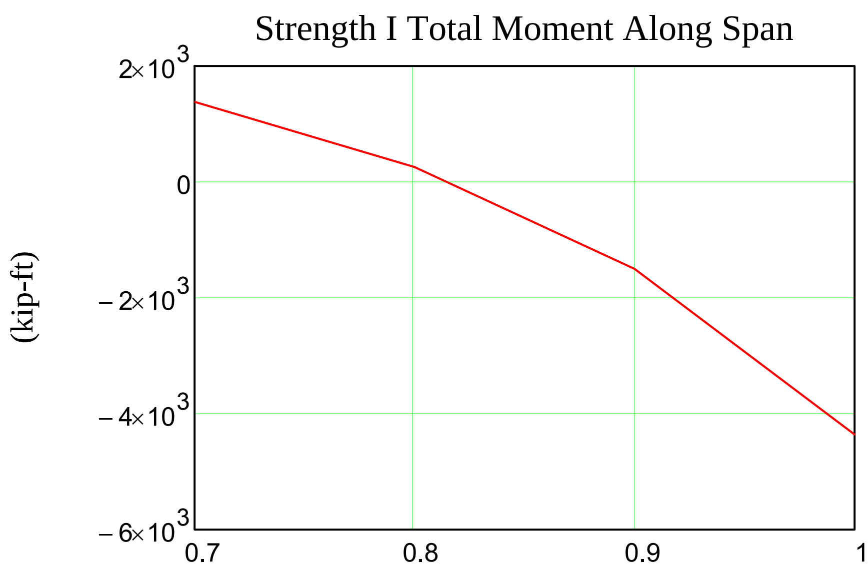
$spa' := spa \cdot 2$ $\boxed{spa' = 8.50}$ in

$As' := \frac{As}{2}$ $\boxed{As' = 10.588}$ in^2

$a' := \frac{As' \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$ $\boxed{a' = 3.11}$ in

$M_{n'} := As' \cdot f_y \cdot \left(d_e - \frac{a'}{2}\right) \cdot \frac{1}{12}$ $\boxed{M_{n'} = 3111}$ kip-ft

$M_{r'} := \phi_f \cdot M_{n'}$ $\boxed{M_{r'} = 2799}$ kip-ft



Based on the moment diagram, try locating the first cut off at $cut_1 := 0.90$ span. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 2799}$ kip-ft

$\boxed{Mu_{cut1} = 1501}$ kip-ft

$\boxed{Ms_{cut1} = 1565}$ kip-ft

Is Mu_{cut1} less than $M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 1709}$ kip-ft

$\boxed{1.33 \cdot Mu_{cut1} = 1996}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and $1.33{*}Mu_{cut1}$? $\boxed{\text{check} = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho' := \frac{As'}{b_w \cdot d_e} \qquad \boxed{\rho' = 0.00585}$$

$$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n \qquad \boxed{k' = 0.206}$$

$$j' := 1 - \frac{k'}{3} \qquad \boxed{j' = 0.931}$$

$\boxed{Ms_{cut1} = 1565}$ kip-ft

$$f_{s'} := \frac{Ms_{cut1}}{As' \cdot j' \cdot d_e} \cdot 12 \quad \le 0.6\ f_y \qquad \boxed{f_{s'} = 31.582} \text{ ksi} \quad \le 0.6\ f_y \text{ O.K.}$$

$\boxed{\beta = 1.076}$

$\boxed{\gamma_e = 0.750}$

$$S_{max'} := \frac{700 \gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c \qquad \boxed{S_{max'} = 9.08} \text{ in}$$

$\boxed{spa' = 8.50}$ in

Is the bar spacing less than $S_{max'}$? $\boxed{\text{check} = \text{"OK"}}$

Check the Fatigue 1 reinforcement limits in accordance with **LRFD [5.5.3]**:

The factored moments at the cut off are: $\boxed{Mf_{DLcut1} = 298}$ kip-ft

$\boxed{Mf_{LLcut1} = 458}$ kip-ft

$\boxed{Mfpos_{LLcut1} = 140}$ kip-ft

$$Mf_{cut1} := Mf_{DLcut1} + Mf_{LLcut1} \qquad \boxed{Mf_{cut1} = 756} \text{ kip-ft}$$

Check stress in section for determination of use of cracked or uncracked section properties:

$$f_{top\_cut1} := \frac{Mf_{cut1}}{S_c} \cdot 12 \qquad \boxed{f_{top\_cut1} = 0.234} \text{ ksi}$$

$$f_{limit} := 0.095 \cdot \sqrt{f'_c} \qquad \boxed{f_{limit} = 0.269} \text{ ksi}$$

Therefore: $\boxed{\text{SectionProp} = \text{"Un-Cracked"}}$

$f_{min\_cut1} := n \cdot \frac{Mf_{cut1}}{S_c} \cdot 12$ $\boxed{f_{min\_cut1} = 1.066}$ ksi

$\Delta F_{TH\_cut1} := 24 - 0.33 \cdot f_{min\_cut1}$ (for f_y = 60 ksi) $\boxed{\Delta F_{TH\_cut1} = 23.648}$ ksi

The live load range is the sum of the positive and negative fatigue moments:

$Mf_{LLrange} := Mf_{LLcut1} + Mfpos_{LLcut1}$ $\boxed{Mf_{LLrange} = 598}$ kip-ft

$\gamma fLL\Delta f\_cut1 := n \cdot \frac{Mf_{LLrange}}{S_c} \cdot 12$ $\boxed{\gamma fLL\Delta f\_cut1 = 0.844}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{\text{check} = \text{"OK"}}$

Therefore this cut off location, $\boxed{cut_1 = 0.90}$, is OK. The bar shall be extended past the cut off point a distance not less than the maximum of the following, **LRFD [5.11.1.2.3]**:

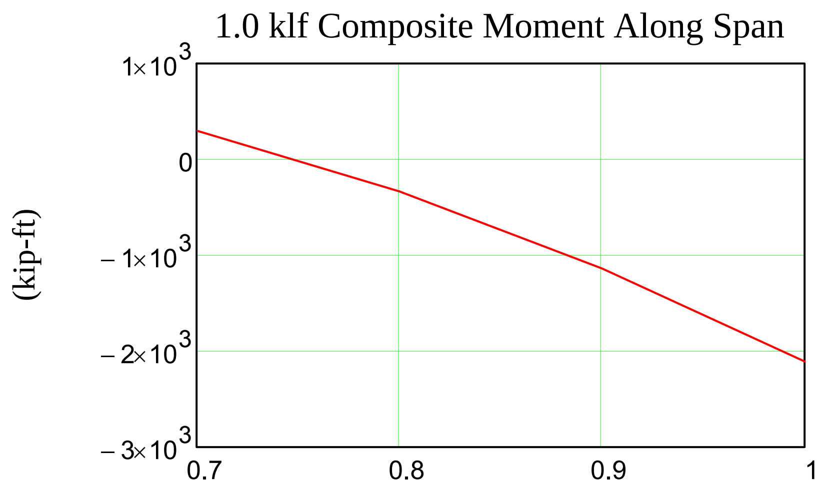
$$extend := \begin{pmatrix} d_e \\ 12 \cdot Bar_D(Bar_{No}) \\ 0.0625 \cdot L \cdot 12 \end{pmatrix} \qquad \boxed{extend = \begin{pmatrix} 60.311 \\ 13.536 \\ 97.500 \end{pmatrix}}$$

$\boxed{\frac{\max(extend)}{12} = 8.13}$ ft

$X_1 := L \cdot (1 - cut_1) + \frac{\max(extend)}{12}$ $\boxed{X_1 = 21.12}$ feet

USE $\boxed{X_1 = 22}$ feet from the CL of the pier.

The second bar cut off is located at the point of inflection under a uniform 1.0 klf composite dead load. At $cut_2 = 0.750$, $M_{cut2} = (79)$ kip-ft. Extend the bar the max(extend) distance calculated above past this point, or 4 feet past the first cut off, whichever is greater.



$X_{2a} := L \cdot (1 - cut_2) + \frac{max(extend)}{12}$ $\boxed{X_{2a} = 40.63}$ feet from the center of the pier

$X_{2b} := X_1 + 4$ $\boxed{X_{2b} = 26.00}$ feet from the center of the pier

$X_2 := max(X_{2a}, X_{2b})$ $\boxed{X_2 = 40.63}$ feet

USE $\boxed{X_2 = 41}$ feet from the CL of the pier.

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E19-3 Box Section Beam

This example shows design calculations for a single span prestressed box multi-beam bridge having a 2" concrete overlay and is designed for a 20 pound per square foot future wearing surface. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2016 Interim. Note: Example has not been updated to current Bridge Manual guidance and should be used for informational purposes only)***

E19-3.1 Preliminary Structure Data

Design Data
A-1 Abutments at both ends
Skew: 0 degrees
Live Load: HL-93
Roadway Width: 28 ft. minimum clear

$L := 44$	Span Length, single span, ft
$L_g := 44.5$	Girder Length, the girder extends 3" past the CL bearing at each abutment, single span, ft
$N_L := 2$	Number of design lanes
$t_{overlay} := 2$	Minimum overlay thickness, inches
$f_{pu} := 270$	Ultimate tensile strength for low relaxation strands, ksi
$d_s := 0.5$	Strand diameter, inches
$A_s := 0.1531$	Area of prestressing strands, in^2
$E_s := 28500$	Modulus of elasticity of the prestressing strands, ksi
$f'_c := 5$	Concrete strength (prestressed box girder), ksi
$f'_{ci} := 4.25$	Concrete strength at release, ksi
$K_1 := 1.0$	Aggregate correction factor
$w_c := 0.150$	Unit weight of concrete for box girder, overlay, and grout, kcf
$fy := 60$	Bar steel reinforcement, Grade 60, ksi.
$w_{rail} := 0.075$	Weight of Type "M" rail, klf
$Wh_{rail} := 0.42$	Width of horizontal members of Type "M" rail, feet
$\mu := 0.20$	Poisson's ratio for concrete, **LRFD [5.4.2.5]**

Based on past experience, the modulus of elasticity for the precast concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi for a concrete strength of 6 ksi. The values of E for different concrete strengths are calculated as follows:

$$E_{beam5} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}}$$ $\boxed{E_{beam5} = 5021}$ ksi

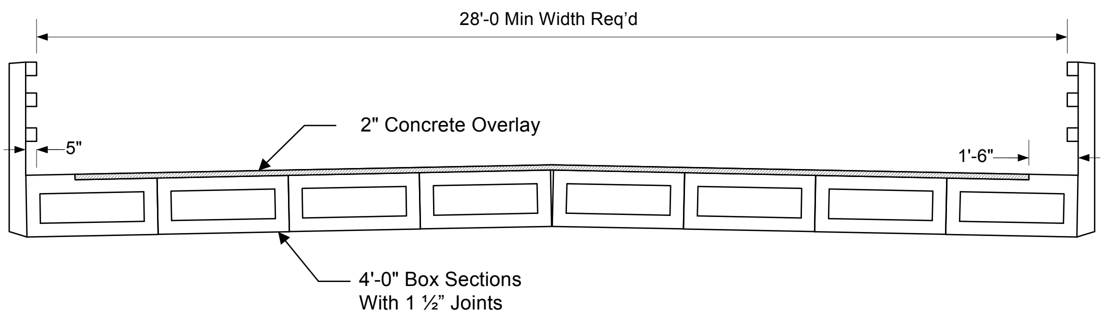
$E_B := E_{beam5}$

The modulus of elasticity at the time of release is calculated in accordance with **LRFD [C5.4.2.4]**.

$$E_{beam4.25} := 33000 \cdot K_1 \cdot w_c^{\,1.5} \cdot \sqrt{f'_{ci}}$$ $\boxed{E_{beam4.25} = 3952}$ ksi

$E_{ct} := E_{beam4.25}$

Based on the preliminary data, Section 19.3.9 of this chapter and Table 19.3-3, select a 4'-0" wide pretensioned box section having a depth of 1'-9" (Section 3), as shown on Bridge Manual Standard 19.15. The actual total deck width provided is calculated below.



$n_{beams} := 8$

$n_{joints} := n_{beams} - 1$ $\boxed{n_{joints} = 7}$

$W_s := 4$ Width of section, ft

$W_j := 1.5$ Width of joints, inches

Overall width of the bridge, ft

$$W_b := n_{beams} \cdot W_s + n_{joints} \cdot \frac{W_j}{12}$$ $\boxed{W_b = 32.875}$ feet

Clear width of the bridge, ft

$W_{b\_clear} := W_b - 2 \cdot Wh_{rail}$ $\boxed{W_{b\_clear} = 32.035}$ feet

$W_{curb} := 1.5$ Width of curb on exterior girder (for steel rails), feet

$S := W_s + \frac{W_j}{12}$ Effective spacing of sections $\boxed{S = 4.125}$ feet

<u>Section Properties, 4 ft x 1'-9" deep Box, Section 3</u>

$D_s := 1.75$ Depth of section, ft

$A := 595$ Area of the box girder, in^2

$t_w := 5$ Thickness of each vertical element, in

$r_{sq} := 55.175$ in^2

$y_t := 10.5$ in

$y_b := -10.5$ in

$S_t := 3137$ Section modulus, in^3

$S_b := -3137$ Section modulus, in^3

$I := 32942$ Moment of inertia, in^4

$J := 68601$ St. Venant's torsional inertia, in^4

E19-3.2 Live Load Distribution

The live load distribution for adjacent box beams is calculated in accordance with **LRFD [4.6.2.2.2]**. Note that if the section does not fall within the applicability ranges, the lever rule shall be used to determine the distribution factor.

E19-3.2.1 Distribution for Moment

For interior beams, the live load moment distribution factor is calculated as indicated in **LRFD [Table 4.6.2.2.2b-1]** for cross section type "g" if connected only enough to prevent relative vertical displacement. This distribution factor applies regardless of the number of lanes loaded.

$K := \sqrt{\frac{(1+\mu)\cdot I}{J}}$ $\boxed{K = 0.759}$

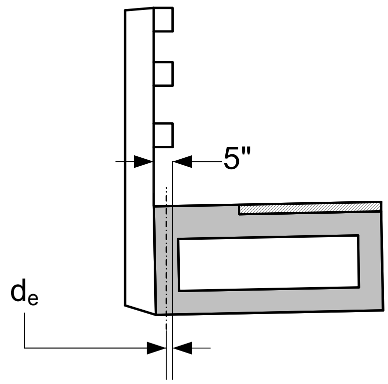
$C := \min\left[K\cdot\left(\frac{W_b}{L}\right), K\right]$ $\boxed{C = 0.567}$

When C is less than 5:

$D := 11.5 - N_L + 1.4\cdot N_L\cdot(1 - 0.2\cdot C)^2$ $\boxed{D = 11.701}$

$g_{int\_m} := \frac{S}{D}$ $\boxed{g_{int\_m} = 0.353}$

For exterior beams, the live load moment distribution factor is calculated as indicated in **LRFD [Table 4.6.2.2.2d-1]** for cross section type "g".



$d_e := \frac{5}{12} \cdot \frac{1}{2} - Wh_{rail}$

Distance from the center of the exterior web to the face of traffic barrier, ft.

$\boxed{d_e = -0.212}$ feet

For one design lane loaded:

$e_1 := \max\left(1.125 + \frac{d_e}{30}, 1\right)$ $\boxed{e_1 = 1.118}$

$g_{ext1} := g_{int\_m} \cdot e_1$ $\boxed{g_{ext1} = 0.394}$

For two or more design lanes loaded:

$e_2 := \max\left(1.04 + \frac{d_e}{25}, 1\right)$ $\boxed{e_2 = 1.032}$

$g_{ext2} := g_{int\_m} \cdot e_2$ $\boxed{g_{ext2} = 0.364}$

Use the maximum value from the above calculations to determine the controlling exterior girder distribution factor for moment.

$\boxed{g_{ext\_m} := \max(g_{ext1}, g_{ext2})}$ $\boxed{g_{ext\_m} = 0.394}$

The distribution factor for fatigue is the single lane distribution factor with the multi-presence factor, $m := 1.2$, removed:

$g_f := \frac{g_{ext1}}{1.2}$ $\boxed{g_f = 0.328}$

E19-3.2.2 Distribution for Shear

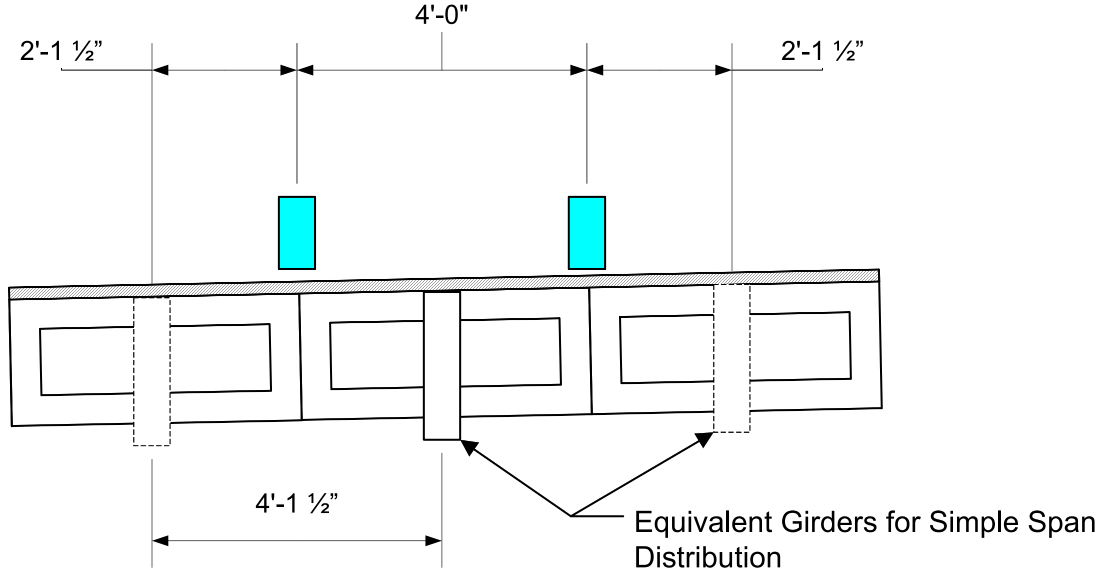
Interior Girder

This section does not fall in the range of applicability for shear distribution for interior girders of bridge type "g". I = 32942 in^4 and the limit is 40000 < I < 610,000, per **LRFD [Table 4.6.2.2.3a-1]**. Therefore, use the lever rule.

For the single lane loaded, only one wheel can be located on the box section. With the single lane multi presence factor, the interior girder shear distribution factor is:

$g_{int\_v1} := 0.5 \cdot 1.2$ $\boxed{g_{int\_v1} = 0.600}$

For two or more lanes loaded, center adjacent vehicles over the beam. One load from each vehicle acts on the beam.



$g_{int\_v2} := 0.5 \cdot \dfrac{2.125}{4.125} \cdot 2$ $\boxed{g_{int\_v2} = 0.515}$

$g_{int\_v} := \max\left(g_{int\_v1}, g_{int\_v2}\right)$ $\boxed{g_{int\_v} = 0.600}$

Exterior Girder

For the exterior girder, the range of applicability of **LRFD [T-4.6.2.2.3b-1]** for bridge type "g" is satisfied.

For the single lane loaded:

$e_{v1} := \max\left(1.25 + \dfrac{d_e}{20}, 1.0\right)$ $\boxed{e_{v1} = 1.239}$

$g_{ext\_v1} := e_{v1} \cdot g_{int\_v1}$ $\boxed{g_{ext\_v1} = 0.744}$

For two or more lanes loaded:

$b := W_s \cdot 12$ $\boxed{b = 48}$ inches

$$e_{v2} := \max\left[1 + \left(\frac{d_e + \frac{b}{12} - 2.0}{40}\right)^{0.5}, 1.0\right]$$ $\boxed{e_{v2} = 1.211}$

$g_{ext\_v2} := e_{v2} \cdot g_{int\_v2}$ $\boxed{g_{ext\_v2} = 0.624}$

$g_{ext\_v} := \max(g_{ext\_v1}, g_{ext\_v2})$ $\boxed{g_{ext\_v} = 0.744}$

E19-3.3 Live Load Moments

The HL-93 live load moment per lane on a 44 foot span is controlled by the design tandem plus lane. The maximum value at mid-span, including a dynamic load allowance of 33%, is $M_{LL\_lane} := 835.84$ kip-ft per lane.

$M_{LLint} := M_{LL\_lane} \cdot g_{int\_m}$ $\boxed{M_{LLint} = 294.7}$ kip-ft

$M_{LLext} := M_{LL\_lane} \cdot g_{ext\_m}$ $\boxed{M_{LLext} = 329.4}$ kip-ft

The Fatige live load moment per lane on a 44 foot span at mid-span, including a dynamic load allowance of 15%, is $M_{LLfat\_lane} := 442.4$ kip-ft per lane.

$M_{LLfat} := M_{LLfat\_lane} \cdot g_f$ $\boxed{M_{LLfat} = 145.3}$ kip-ft

E19-3.4 Dead Loads

Interior Box Girders

Box Girder $w_g := \frac{A}{12^2} \cdot w_c$ $\boxed{w_g = 0.620}$ klf

Internal Concrete Diaphragm (at center of span)

$$w_{diaph} := 1.17 \cdot \left(D_s - \frac{10}{12}\right) \cdot \left(W_s - \frac{10}{12}\right) \cdot w_c \qquad \boxed{w_{diaph} = 0.509} \quad \text{kips}$$

Equivalent uniform load:

$$w_{d\_mid} := 2 \cdot \frac{w_{diaph}}{L} \qquad \boxed{w_{d\_mid} = 0.023} \quad \text{klf}$$

Internal Concrete Diaphragm (at ends of span)

$$w_{diaph\_end} := 2.83 \cdot \left(D_s - \frac{10}{12}\right) \cdot \left(W_s - \frac{10}{12}\right) \cdot w_c \qquad \boxed{w_{diaph\_end} = 1.232} \quad \text{kips}$$

Equivalent uniform load:

$$w_{d\_end} := 8 \cdot \frac{w_{diaph\_end} \cdot 1.17}{L^2} \qquad \boxed{w_{d\_end} = 0.006} \quad \text{klf}$$

$$w_d := w_{d\_mid} + w_{d\_end} \qquad \boxed{w_d = 0.029} \quad \text{klf}$$

For the interior girders, all dead loads applied after the post tensioning has been completed are distributed equally to all of the girders.

Overlay

$$w_o := \frac{\frac{t_{overlay}}{12} \cdot \left(W_b - W_{curb} \cdot 2\right) \cdot w_c}{n_{beams}} \qquad \boxed{w_o = 0.093} \quad \text{klf}$$

Joint Grout

$$w_j := \frac{W_j}{12} \cdot \left(D_s + \frac{t_{overlay}}{12}\right) \cdot w_c \cdot \frac{n_{joints}}{n_{beams}} \qquad \boxed{w_j = 0.031} \quad \text{klf}$$

"M" Rail

$$w_r := \frac{2 \cdot w_{rail}}{n_{beams}} \qquad \boxed{w_r = 0.019} \quad \text{klf}$$

Future Wearing Surface

$$w_{fws} := \frac{W_b \cdot 0.020}{n_{beams}} \qquad \boxed{w_{fws} = 0.082} \quad \text{klf}$$

$$w_{DCint} := w_g + w_d + w_o + w_j + w_r \qquad \boxed{w_{DCint} = 0.792} \quad \text{klf}$$

$w_{DWint} := w_{fws}$ $\boxed{w_{DWint} = 0.082}$ klf

Exterior Box Girders

Box Girder $w_{g\_ext} := \frac{A + 2 \cdot W_{curb} \cdot 12}{12^2} \cdot w_c$ $\boxed{w_{g\_ext} = 0.657}$ klf

Internal Concrete Diaphragms $\boxed{w_d = 0.029}$ klf

For the exterior girders, all dead loads applied directly to the girder are applied.

Overlay $w_{o\_ext} := \frac{t_{overlay}}{12} \cdot (S - W_{curb}) \cdot w_c$ $\boxed{w_{o\_ext} = 0.066}$ klf

Joint Grout $w_{j\_ext} := \frac{1}{2} \cdot \frac{W_j}{12} \cdot \left(D_s + \frac{t_{overlay}}{12}\right) \cdot w_c$ $\boxed{w_{j\_ext} = 0.018}$ klf

Type M Rail $w_{r\_ext} := w_{rail}$ $\boxed{w_{r\_ext} = 0.075}$ klf

Future Wearing Surface

$w_{fws\_ext} := S \cdot 0.020$ $\boxed{w_{fws\_ext} = 0.083}$ klf

$w_{DCext} := w_{g\_ext} + w_d + w_{o\_ext} + w_{j\_ext} + w_{r\_ext}$

$\boxed{w_{DCext} = 0.845}$ klf

$w_{DWext} := w_{fws\_ext}$ $\boxed{w_{DWext} = 0.083}$ klf

E19-3.5 Dead Load Moments

Moment of the girder and internal diaphragms alone, based on total girder length.

$M_{gi} := (w_g + w_d) \cdot \frac{L_g^{\,2}}{8}$ $\boxed{M_{gi} = 160.6}$ kip-ft

$M_{gext} := (w_{g\_ext} + w_d) \cdot \frac{L_g^{\,2}}{8}$ $\boxed{M_{gext} = 169.9}$ kip-ft

Interior Girder

$M_{DCint} := w_{DCint} \cdot \frac{L^2}{8}$ $\boxed{M_{DCint} = 191.8}$ kip-ft

$M_{DWint} := w_{DWint} \cdot \frac{L^2}{8}$ $\boxed{M_{DWint} = 19.9}$ kip-ft

Exterior Girder

$M_{DCext} := w_{DCext} \cdot \frac{L^2}{8}$ $\boxed{M_{DCext} = 204.5}$ kip-ft

$M_{DWext} := w_{DWext} \cdot \frac{L^2}{8}$ $\boxed{M_{DWext} = 20.0}$ kip-ft

E19-3.6 Design Moments

Calculate the total moments on the interior and exterior girders to determine which girder will control the design.

$M_{T\_int} := M_{DCint} + M_{DWint} + M_{LLint}$ $\boxed{M_{T\_int} = 506.3}$ kip-ft

$M_{T\_ext} := M_{DCext} + M_{DWext} + M_{LLext}$ $\boxed{M_{T\_ext} = 553.9}$ kip-ft

Since the Dead Load moments are very close and the exterior Live Load moments are greater than the interior moments, the exterior girder controls for this design example. Note: an interior box girder section design will not be provided in this example. However, the interior girder shall not have less load carrying capacity then the exterior girder.

$M_{DC} := M_{DCext}$ $\boxed{M_{DC} = 204.5}$ kip-ft

$M_{DW} := M_{DWext}$ $\boxed{M_{DW} = 20}$ kip-ft

$M_{LL} := M_{LLext}$ $\boxed{M_{LL} = 329.4}$ kip-ft

$M_{LLf} := M_{LLfat}$ $\boxed{M_{LLf} = 145.3}$ kip-ft

E19-3.7 Load Factors

From **LRFD [Table 3.4.1-1 & Table 3.4.1-4]**:

	DC	DW	LL
Strength 1	$\gamma st_{DC} := 1.25$	$\gamma st_{DW} := 1.50$	$\gamma st_{LL} := 1.75$
Service 1	$\gamma s1_{DC} := 1.0$	$\gamma s1_{DW} := 1.0$	$\gamma s1_{LL} := 1.0$
Service 3	$\gamma s3_{DC} := 1.0$	$\gamma s3_{DW} := 1.0$	$\gamma s3_{LL} := 0.8$
Fatigue 1			$\gamma f_{LL} := 1.5$

E19-3.8 Factored Moments

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored moments for each limit state are calculated by applying the appropriate load factor to the girder moments. For the exterior girder:

Strength 1

$$M_{str} := \eta \cdot (\gamma st_{DC} \cdot M_{DC} + \gamma st_{DW} \cdot M_{DW} + \gamma st_{LL} \cdot M_{LL})$$

$$= 1.0 \cdot (1.25 \cdot M_{DC} + 1.50 \cdot M_{DW} + 1.75 \cdot M_{LL}) \qquad \boxed{M_{str} = 862} \quad \text{kip-ft}$$

Service 1 (for compression checks)

$$M_{s1} := \eta \cdot (\gamma s1_{DC} \cdot M_{DC} + \gamma s1_{DW} \cdot M_{DW} + \gamma s1_{LL} \cdot M_{LL})$$

$$= 1.0 \cdot (1.0 \cdot M_{DC} + 1.0 \cdot M_{DW} + 1.0 \cdot M_{LL}) \qquad \boxed{M_{s1} = 554} \quad \text{kip-ft}$$

Service 3 (for tension checks)

$$M_{s3} := \eta \cdot (\gamma s3_{DC} \cdot M_{DC} + \gamma s3_{DW} \cdot M_{DW} + \gamma s3_{LL} \cdot M_{LL})$$

$$= 1.0 \cdot (1.0 \cdot M_{DC} + 1.0 \cdot M_{DW} + 0.8 \cdot M_{LL}) \qquad \boxed{M_{s3} = 488} \quad \text{kip-ft}$$

Fatigue 1 (for compression checks)

$$M_{f1} := \eta \cdot \left[\frac{1}{2} \cdot (M_{DC} + M_{DW}) + \gamma f_{LL} \cdot M_{LLf}\right]$$

$$= 1.0 \cdot \left[\frac{1}{2} \cdot (M_{DC} + M_{DW}) + 1.5 \cdot M_{LLf}\right] \qquad \boxed{M_{f1} = 330} \quad \text{kip-ft}$$

E19-3.9 Allowable Stress

Allowable stresses are determined for 2 sages for prestressed girders. Temporary allowable stresses are set for the loading stage at release of the prestressing strands. Final condition allowable stresses are checked at the end of 50 years of service.

E19-3.9.1 Temporary Allowable Stresses

The temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

$f_{ciall} := 0.65 \cdot f'_{ci}$ $\boxed{f_{ciall} = 2.763}$ ksi

In accordance with **LRFD [Table 5.9.4.1.2-1]**, the temporary allowable tension stress is calculated as follows (assume there is no bonded reinforcement):

$f_{tiall} := -\min\left(0.0948 \cdot \lambda \sqrt{f'_{ci}}, 0.2\right)$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tiall} = -0.195}$ ksi

If bonded reinforcement is present in the top flange, the temporary allowable tension stress is calculated as follows:

$f_{tiall\_bond} := -0.24 \cdot \lambda \sqrt{f'_{ci}}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tiall\_bond} = -0.495}$ ksi

E19-3.9.2 Final Condition Allowable Stresses

Allowable Stresses,**LRFD [5.9.4.2.1]**:

There are two compressive service stress limits:

$f_{call1} := 0.45 \cdot f'_c$ PS + DL $\boxed{f_{call1} = 2.250}$ ksi

$f_{call2} := 0.60 \cdot f'_c$ LL + PS + DL $\boxed{f_{call2} = 3.000}$ ksi

There is one tension service stress limit **LRFD [5.9.4.2.2]**:

$f_{tall} = -0.19 \cdot \lambda \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$f_{tall} := -0.19 \cdot \sqrt{f'_c}$ LL + PS + DL $|f_{tall}| \le 0.6$ ksi $\boxed{f_{tall} = -0.425}$ ksi

There is one compressive fatigue stress limit **LRFD [5.5.3.1]**:

$f_{call\_f} := 0.40 \cdot f'_c$ LLf + 1/2(PS + DL) $\boxed{f_{call\_f} = 2.000}$ ksi

E19-3.10 Preliminary Design Steps

The following steps are utilized to design the prestressing strands:

1) Design the amount of prestress to prevent tension at the bottom of the beam under the full load at center span after 50 years.

2) Calculate the prestress losses and check the girder stresses at mid span at the time of transfer.

3) Check resulting stresses at the critical sections of the girder at the time of transfer and after 50 years.

E19-3.10.1 Determine Amount of Prestress

Design the amount of prestress to prevent tension at the bottom of the beam under the full loac (at center span) after 50 years.

Near center span, after 50 years, T = the remaining effective prestress, aim for no tension at the bottom. Use Service I for compression and Service III for tension.

For this example, the exterior girder has the controlling moments.

Calculate the stress at the bottom of the beam due to the Service 3 loading:

$$f_b := \frac{M_{s3} \cdot 12}{S_b} \qquad \boxed{f_b = -1.867} \text{ ksi}$$

Stress at bottom due to prestressing:

$$f_{bp} = \frac{T}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

and $f_{bp} := |f_b|$ desired final prestress.

We want this to balance out the tensile stress calculated above from the loading, i.e. an initial compression. The required stress due to prestress force at bottom of section to counteract the Service 3 loads:

$$\boxed{f_{bp} = 1.867} \text{ ksi}$$

E19-3.10.1.1 Estimate the Prestress Losses

At 50 years the prestress has decreased (due to CR, SH, RE):

The approximate method of estimated time dependent losses is used by WisDOT. The lump sum loss estimate, I-girder loss **LRFD [Table 5.9.5.3-1]**

Where PPR is the partial prestressing ratio, $PPR := 1.0$

$F_{delta} := 26 + 4 \cdot PPR$ $\boxed{F_{delta} = 30}$ ksi

but, for low relaxation strand: $F_{Delta} := F_{delta} - 6$ $\boxed{F_{Delta} = 24}$ ksi

Assume an initial strand stress; $f_{tr} := 0.75 \cdot f_{pu}$ $\boxed{f_{tr} = 202.5}$ ksi

Based on experience, assume $\Delta f_{pES\_est} := 9.1$ ksi loss from elastic shortening. As an alternate initial estimate, **LRFD [C.5.9.5.2.3a]** suggests assuming a 10% ES loss.

$$ES_{loss} := \frac{\Delta f_{pES\_est}}{f_{tr}} \cdot 100$$ $\boxed{ES_{loss} = 4.494}$ %

$f_i := f_{tr} - \Delta f_{pES\_est}$ $\boxed{f_i = 193.4}$ ksi

The total loss is the time dependant losses plus the ES losses:

$loss := F_{Delta} + \Delta f_{pES\_est}$ $\boxed{loss = 33.1}$ ksi

$$loss_{\%} := \frac{loss}{f_{tr}} \cdot 100$$ $\boxed{loss_{\%} = 16.346}$ % (estimated)

If T_o is the initial prestress, then (1-loss)*T_o is the remaining:

$$T = (1 - loss_{\%}) \cdot T_o$$

$$ratio := 1 - \frac{loss_{\%}}{100}$$ $\boxed{ratio = 0.837}$

$$T = ratio \cdot T_o$$

$$f_{bp} = \frac{(1 - loss_{\%}) \cdot T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

OR:

$$\frac{f_{bp}}{1 - loss_{\%}} = \frac{T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

$$f_{bpi} := \frac{f_{bp}}{1 - \frac{loss_{\%}}{100}}$$

$\boxed{f_{bpi} = 2.231}$ ksi

desired bottom initial prestress

E19-3.10.1.2 Determine Number of Strands

$A_s = 0.153$ in^2

$f_{pu} = 270$ ksi

$f_s := 0.75 \cdot f_{pu}$ $\boxed{f_s = 202.5}$ ksi

$P := A_s \cdot f_s$ $\boxed{P = 31.003}$ kips per strand

$$f_{bp} := \frac{P \cdot N}{A} \cdot \left(1 + e \cdot \frac{y_b}{r_{sq}}\right)$$

$y_b = -10.5$ Distance from the centroid of the 21" depth to the bottom of the box section, in.

For the 4'-0 wide box sections, there can be up to 22 strands in the bottom row and 2 rows of strands in the sides of the box. Calculate the eccentricity for the maximum number of strands that can be placed in the bottom row of the box:

$e_b := y_b + 2$ $e_b = -8.5$ Eccentricity to the bottom row of strands, inches

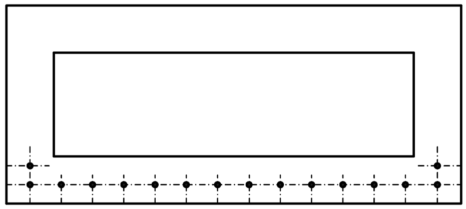
$e_s := e_b$ $\boxed{e_s = -8.5}$ inches

$$N_{req} := \frac{f_{bpi} \cdot A}{P} \cdot \frac{1}{1 + e_s \cdot \frac{y_b}{r_{sq}}}$$

$\boxed{N_{req} = 16.4}$ strands

Therefore, try $N := 16$ strands since some final tension in the bottom of the girder is allowed.

Place 2 of the strands in the second row:



$$e_s := \frac{e_b \cdot 14 + (e_b + 2) \cdot 2}{16}$$

$\boxed{e_s = -8.25}$ inches

E19-3.10.2 Prestress Loss Calculations

The loss in prestressing force is comprised of the following components:

1) Elastic Shortening (ES), shortening of the beam as soon as prestress is applied. Can this be compensated for by overstressing?

2) Shrinkage (SH), shortening of the concrete as it hardens, time function.

3) Creep (CR), slow shortening of concrete due to permanent compression stresses in the beam, time function.

4) Relaxation (RE), the tendon slowly accommodates itself to the stretch and the internal stress drops with time

E19-3.10.2.1 Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$T_{oi} := N \cdot f_{tr} \cdot A_s$ $\boxed{= 16 \cdot 0.75 \cdot 270 \cdot 0.1531 = 496}$ kips

The ES loss estimated above was: $\Delta f_{pES\_est} = 9.1$ ksi, or $ES_{loss} = 4.494$ %. The resulting force in the strands after ES loss:

$$T_o := \left(1 - \frac{ES_{loss}}{100}\right) \cdot T_{oi}$$

$\boxed{T_o = 474}$ kips

Since all strands are straight, we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A} + (T_o \cdot e_s) \cdot \frac{e_s}{I} + M_{gi} \cdot 12 \cdot \frac{e_s}{I}$$

$\boxed{f_{cgp} = 1.264}$ ksi

$\boxed{E_{ct} = 3952}$ ksi

$E_p := E_s$ $\boxed{E_p = 28500}$ ksi

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp}$$ $\boxed{\Delta f_{pES} = 9.118}$ ksi

This value of Δf_{pES} is in agreement with the estimated value above; $\Delta f_{pES\_est} = 9.10$ ksi. If these values did not agree, T_o would have to be recalculated using f_{tr} minus the new value of Δf_{pES}, and a new value of f_{cgp} would be determined. This iteration would continue until the assumed and calculated values of Δf_{pES} are in agreement.

The initial stress in the strand is:

$$f_i := f_{tr} - \Delta f_{pES}$$ $\boxed{f_i = 193.382}$ ksi

The force in the beam after transfer is:

$$T_o := N \cdot A_s \cdot f_i$$ $\boxed{T_o = 474}$ kips

Check the design to avoid premature failure at the center of the span at the time of transfer. Check the stress at the center span (at the plant) at both the top and bottom of the girder.

$$f_{ttr} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{gi} \cdot 12}{S_t}$$ $\boxed{f_{ttr} = 0.200}$ ksi

$$f_{btr} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{gi} \cdot 12}{S_b}$$ $\boxed{f_{btr} = 1.392}$ ksi

temporary allowable stress (tension) $\boxed{f_{tiall} = -0.195}$ ksi

temporary allowable stress (compression) $\boxed{f_{ciall} = 2.763}$ ksi

Is the stress at the top of the girder less than the allowable? $\boxed{check = \text{"OK"}}$

Is the stress at the bottom of the girder less than the allowable? $\boxed{check = \text{"OK"}}$

E19-3.10.2.2 Approximate Estimate of Time Dependant Losses

Calculate the components of the time dependant losses; shrinkage, creep and relaxation, using the approximate method in accordance with **LRFD [5.9.5.3]**.

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pi} \cdot A_s}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

From **LRFD [Figure 5.4.2.3.3-1]**, the average annual ambient relative humidity, $H := 72$ %.

$\gamma_h := 1.7 - 0.01 \cdot H$ $\quad \boxed{\gamma_h = 0.980}$

$\gamma_{st} := \dfrac{5}{1 + f'_{ci}}$ $\quad \boxed{\gamma_{st} = 0.952}$

$\Delta f_{pR} := 2.4$ ksi for low relaxation strands

$\Delta f_{pCR} := 10.0 \cdot \dfrac{f_{tr} \cdot A_s \cdot N}{A} \cdot \gamma_h \cdot \gamma_{st}$ $\quad \boxed{\Delta f_{pCR} = 7.781}$ ksi

$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$ $\quad \boxed{\Delta f_{pSR} = 11.200}$ ksi

$\Delta f_{pRE} := \Delta f_{pR}$ $\quad \boxed{\Delta f_{pRE} = 2.400}$ ksi

$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$ $\quad \boxed{\Delta f_{pLT} = 21.381}$ ksi

The total estimated prestress loss (Approximate Method):

$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$ $\quad \boxed{\Delta f_p = 30.499}$ ksi

$\boxed{\dfrac{\Delta f_p}{f_{tr}} \cdot 100 = 15.061}$ % total prestress loss

This value is less than but in general agreement with the initial estimated $loss_{\%} = 16.3$.

The remaining stress in the strands and total force in the beam after all losses is:

$f_{pe} := f_{tr} - \Delta f_p$ $\quad \boxed{f_{pe} = 172.00}$ ksi

$T := N \cdot A_s \cdot f_{pe}$ $\quad \boxed{T = 421}$ kips

E19-3.10.3 Check Stresses at Critical Locations

Check the girder stresses at the end of the transfer length of the strands at release:

Minimum moment on section = girder moment at the plant

$$M_{gz} = \frac{w_g}{2} \cdot \left(L_g \cdot z - z^2\right)$$

Stress in the bottom fiber at transfer:

$$f_{bz} = \frac{T_o}{A} + \frac{T_o \cdot e_{sz}}{S_b} + \frac{M_{gz}}{S_b}$$

The transfer length may be taken as:

$l_{tr} := 60 \cdot d_s$ $\boxed{l_{tr} = 30.00}$ in

$x := \frac{l_{tr}}{12}$ $\boxed{x = 2.50}$ feet

The moment at the end of the transfer length due to the girder dead load:

$$M_{gt} := \frac{w_{g\_ext}}{2} \cdot \left(L_g \cdot x - x^2\right) + \left(\frac{w_{diaph} \cdot x}{2} + w_{diaph\_end} \cdot x\right)$$

$\boxed{M_{gt} = 38}$ kip-ft

The girder stresses at the end of the transfer length:

$f_{tt} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_t} + \frac{M_{gt} \cdot 12}{S_t}$ $\boxed{f_{tt} = -0.303}$ ksi

$\boxed{f_{tiall} = -0.195}$ ksi

$\boxed{check = \text{"NG"}}$

If bonded reinforcement is provided in the top flange, the allowable stress is:

$f_{tiall\_bond} = -0.495$ ksi

Is f_{tt} less than f_{tiall_ond}? $\boxed{check = \text{"OK"}}$

$f_{bt} := \frac{T_o}{A} + \frac{T_o \cdot e_s}{S_b} + \frac{M_{gt} \cdot 12}{S_b}$ $\boxed{f_{bt} = 1.896}$ ksi

$\boxed{f_{ciall} = 2.763}$ ksi

Is f_{bt} less than f_{ciall}? $\boxed{check = \text{"OK"}}$

Check final stresses after all losses at the mid-span of the girder:

Top of girder stress (Compression - Service 1):

$f_{t1} := \frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_t}$ PS + DL $\boxed{f_{t1} = 0.459}$ ksi

$\boxed{check = \text{"OK"}}$

$$f_{t2} := \frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{M_{s1} \cdot 12}{S_t}$$ LL + PS + DL $\boxed{f_{t2} = 1.719}$ ksi

$\boxed{\text{check} = \text{"OK"}}$

Bottom of girder stress (Compression - Service 1):

$$f_{b1} := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_b}$$ PS + DL $\boxed{f_{b1} = 0.958}$ ksi

$\boxed{\text{check} = \text{"OK"}}$

Bottom of girder stress (Tension - Service 3):

$$f_b := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{M_{s3} \cdot 12}{S_b}$$ $\boxed{f_b = -0.051}$ ksi

$\boxed{\text{check} = \text{"OK"}}$

Top of girder stress (Compression - Fatigue 1):

$$f_{tf1} := \frac{1}{2} \cdot \left[\frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_t} \right] + \frac{M_{LLf} \cdot 12}{S_t}$$ 1/2(PS + DL) + LLf

$\boxed{f_{t1} = 0.459}$ ksi

$\boxed{\text{check} = \text{"OK"}}$

allowable stress (tension) $\boxed{f_{tall} = -0.425}$ ksi

allowable stress (compression) $\boxed{f_{call1} = 2.250}$ ksi

$\boxed{f_{call2} = 3.000}$ ksi

$\boxed{f_{call\_f} = 2.000}$ ksi

E19-3.11 Flexural Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.7.3.1.1]**:

$\boxed{f_{pe} = 172}$ ksi $\boxed{0.5 \cdot f_{pu} = 135}$ ksi

Is 0.5*f_{pu} less than f_{pe}? $\boxed{check = \text{"OK"}}$

Then at failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD [Table C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assume that the compression block is in the top section of the box. Calculate the capacity as if it is a rectangular section. The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := N \cdot A_s$ $\boxed{A_{ps} = 2.45}$ in^2

$b := W_s \cdot 12$ $\boxed{b = 48.00}$ in

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_c - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.800}$

$d_p := y_t - e_s$ $\boxed{d_p = 18.75}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 3.82}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 3.06}$ in

This is within the depth of the top slab (5-inches). Therefore our assumption is OK.

Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 254.6}$ ksi

$$T_u := f_{ps} \cdot A_{ps}$$ $\boxed{T_u = 624}$ kips

Calculate the nominal moment capacity of the section in accordance with **LRFD [5.7.3.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 895}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$$M_r := \phi_f \cdot M_n$$ $\boxed{M_r = 895}$ kip-ft

The required capacity:

Exterior Girder Moment

$$M_u := M_{str}$$ $\boxed{M_u = 862}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]** for the interior girder:

$\boxed{1.33 \cdot M_u = 1147}$ kip-ft

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.537}$ ksi

$$f_{cpe} := \frac{T}{A} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 1.816}$ ksi

$$S_c := -S_b$$ $\boxed{S_c = 3137}$ ksi

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete structures

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot (\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}) \cdot \frac{1}{12} \right]$$ $\boxed{M_{cr} = 747}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

E19-3.12 Shear Analysis

A separate analysis must be conducted to estimate the total shear force in each girder for shear design purposes.

The live load shear distribution factors to the girders are calculated above in E19-3.2.2.

$\boxed{g_{int\_v} = 0.600}$

$\boxed{g_{ext\_v} = 0.744}$

From section E19-3.4, the uniform dead loads on the girders are:

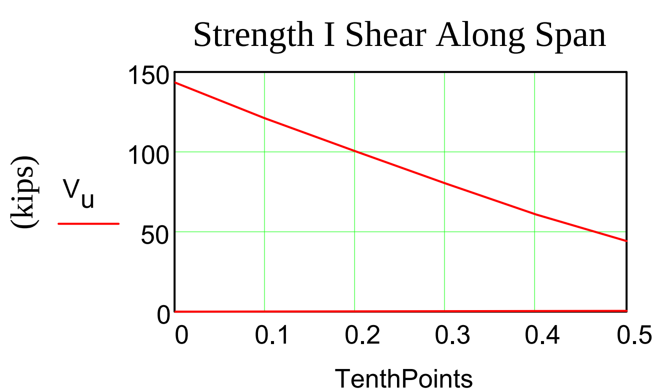
Interior Girder $\boxed{w_{DCint} = 0.792}$ klf

$\boxed{w_{DWint} = 0.082}$ klf

Exterior Girder $\boxed{w_{DCext} = 0.845}$ klf

$\boxed{w_{DWext} = 0.083}$ klf

However, the internal concrete diaphragms were applied as total equivalent uniform loads to determine the maximum mid-span moment. The diaphragm weights should be applied as poin loads for the shear calculations.



$\boxed{Vu_{0.0} = 143.5}$ kips

$\boxed{Vu_{0.5} = 44.2}$ kips

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := 2t_w$ $\boxed{b_v = 10.00}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$d_v := -e_s + y_t - \frac{a}{2}$ $\boxed{d_v = 17.22}$ in

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$L_{crit} := \left(w_{brg} + d_v\right) \cdot \frac{1}{12}$ $\boxed{L_{crit} = 2.10}$ ft

The eccentricity of the strand group at the critical section is:

$\boxed{e_s = -8.25}$ in

Calculation of compression stress block:

$\boxed{d_p = 18.75}$ in

$\boxed{A_{ps} = 2.45}$ in^2

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.0$ for prestressed members with a depth less than 24 inches

$\boxed{d_s = 0.5}$ in

$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_s$ $\boxed{l_d = 70.0}$ in

The transfer length may be taken as: $l_{tr} := 60 \cdot d_s$ $\boxed{l_{tr} = 30.00}$ in

Since $L_{crit} = 2.102$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} \cdot \frac{L_{crit} \cdot 12}{l_{tr}}$$ $\boxed{f_{pu\_crit} = 145}$ ksi

$$T_{crit} := N \cdot A_s \cdot f_{pu\_crit}$$ $\boxed{T_{crit} = 354}$ kips

For rectangular section behavior:

$\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.800}$

$$c_{crit} := \frac{A_{ps} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu\_crit}}{d_p}}$$ $\boxed{c_{crit} = 2.102}$ in

$$a_{crit} := \beta_1 \cdot c_{crit}$$ $\boxed{a_{crit} = 1.682}$ in

Calculation of shear depth based on refined calculations of a:

$$d_{v\_crit} := -e_s + y_t - \frac{a_{crit}}{2}$$ $\boxed{d_{v\_crit} = 17.91}$ in

This value matches the assumed value of d_v above. OK!

$$d_v := d_{v\_crit}$$

The location of the critical section from the end of the girder is:

$$L_{crit} := \left(w_{brg} + d_v\right) \cdot \frac{1}{12}$$ $\boxed{L_{crit} = 2.159}$ ft

The location of the critical section from the center line of bearing at the abutment is:

$$crit := L_{crit} - 0.25$$ $\boxed{crit = 1.909}$ ft

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**).

Note, the value of V_p does not equal zero in the calculation of V_{cw}.

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kips)

V_i = factored shear force at section due to externally applied loads (Live Loads) occurring simultaneously with M_{max} (kips). (Not necessarily equal to V_u.)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in)

M_{max} = maximum factored moment at section due to externally applied loads (Live Loads) (kip-in)

M_{dnc} = total unfactored dead load moment acting on the noncomposite section (kip-ft)

Values for the following moments and shears are at the critical section, $L_{crit} = 2.16$ feet from the end of the girder at the abutment.

$\boxed{V_d = 18.3}$ kips

$\boxed{V_i = 109.5}$ kips

$\boxed{M_{dnc} = 37.3}$ kip-ft

$\boxed{M_{max} = 111.7}$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$\boxed{M_{max} = 1340}$ kip-in

$f_r = -0.20 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := -0.20 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = -0.447}$ ksi

$\boxed{T = 421}$ kips

$$f_{cpe} := \frac{T_{crit}}{A} + \frac{T_{crit} \cdot e_s}{S_b}$$

$\boxed{f_{cpe} = 1.527}$ ksi

$\boxed{M_{dnc} = 37}$ kip-ft

$\boxed{M_{max} = 1340}$ kip-in

$S_c := S_b$ $\boxed{S_c = -3137}$ in^3

$S_{nc} := S_b$ $\boxed{S_{nc} = -3137}$ in^3

$$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12M_{dnc}}{S_{nc}} \right)$$

$\boxed{M_{cre} = 5746}$ kip-in

Calculate V_{ci}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{ci1} := 0.06 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 24.0}$ kips

$V_{ci2} := 0.02 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$ $\boxed{V_{ci2} = 495.9}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 495.9}$ kips

$f_t := \frac{T_{crit}}{A} + \frac{T_{crit} \cdot e_s}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$ $\boxed{f_t = -0.194}$ ksi

$f_b := \frac{T_{crit}}{A} + \frac{T_{crit} \cdot e_s}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$ $\boxed{f_b = 1.384}$ ksi

$\boxed{y_b = -10.50}$ in

$\boxed{h = 21.00}$ in

$f_{pc} := f_b - y_b \cdot \frac{f_t - f_b}{h}$ $\boxed{f_{pc} = 0.595}$ ksi

$V_{p\_cw} := 0$ (no strands are draped) $\boxed{V_{p\_cw} = 0.0}$ kips

Calculate V_{cw}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{cw} := \left(0.06 \cdot \lambda \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 56.0}$ kips

$V_c := \min(V_{ci}, V_{cw})$ $\boxed{V_c = 56.0}$ kips

Calculate the required shear resistance:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$V_{u\_crit} = \gamma st_{DC} \cdot V_{DCnc} + \gamma st_{DW} \cdot V_{DWnc} + \gamma st_{LL} \cdot Vu_{LL}$

$V_n := \frac{V_{u\_crit}}{\phi_v}$ $\boxed{V_n = 147.6}$ kips

Transverse Reinforcing Design at Critical Section:

The required steel capacity:

$V_s := V_n - V_c - V_p$ $\boxed{V_s = 91.6}$ kips

$A_v := 0.40$ in^2 for 2 - #4 rebar

$f_y := 60$ ksi

$\boxed{d_v = 17.91}$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3\cdot\frac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$

$\boxed{\cot\theta = 1.799}$

$$V_s = A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$

LRFD Eq 5.8.3.3-4 reduced per **C5.8.3.3-1** when α = 90 degrees.

$$s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{V_s}$$

$\boxed{s = 8.441}$ in

Check Maximum Spacing, **LRFD [5.8.2.7]**:

$$v_u := \frac{V_{u\_crit}}{\phi_v \cdot b_v \cdot d_v}$$

$\boxed{v_u = 0.824}$ ksi

$\boxed{0.125 \cdot f'_c = 0.625}$

$$s_{max1} := \begin{cases} \min(0.8 \cdot d_v, 24) & \text{if } v_u < 0.125 \cdot f'_c \\ \min(0.4 \cdot d_v, 12) & \text{if } v_u \ge 0.125 \cdot f'_c \end{cases}$$

$\boxed{s_{max1} = 7.16}$ in

Check Minimum Reinforcing, **LRFD [5.8.2.5]**:

$$s_{max2} := \frac{A_v \cdot f_y}{0.0316 \cdot \lambda \sqrt{f'_c} \cdot b_v}$$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$\boxed{s_{max2} = 33.97}$ in

$$s_{max} := \min(s_{max1}, s_{max2})$$

$\boxed{s_{max} = 7.16}$ in

Therefore use a maximum spacing of $s := 7$ inches.

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$

$\boxed{V_s = 110.4}$ kips

Check V_n requirements:

$$Vn_1 := V_c + V_s + V_p$$

$\boxed{Vn_1 = 166}$ kips

$$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$$

$\boxed{Vn_2 = 224}$ kips

$$V_n := \min(Vn_1, Vn_2)$$

$\boxed{V_n = 166}$ kips

$$V_r := \phi_v \cdot V_n$$

$\boxed{V_r = 149.81}$ kips

$\boxed{V_{u\_crit} = 132.85}$ kips

Is V_{u_crit} less than V_r? $\boxed{check = "OK"}$

Web reinforcing is required in accordance with **LRFD [5.8.2.4]** whenever:

$V_u \geq 0.5 \cdot \phi_v \cdot (V_c + V_p)$ (all values shown are in kips)

At critical section from end of girder: $\boxed{V_{u\_crit} = 133}$ $\boxed{0.5 \cdot \phi_v \cdot (V_c + V_p) = 25}$

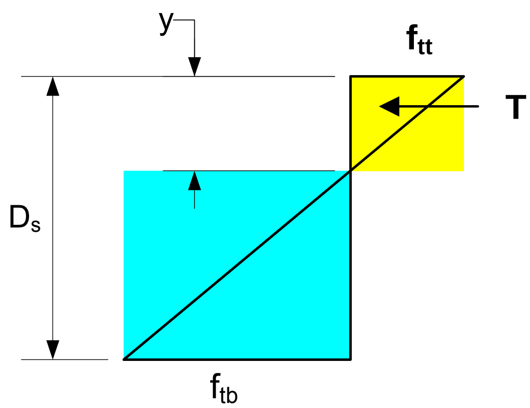
Therefore, use web reinforcing over the entire beam.

Resulting Shear Design:

Use #4 U shaped stirrups at 7-inch spacing between the typical end sections. Unless a large savings in rebar can be realized, use a single stirrup spacing between the standard end sections.

E19-3.13 Non-Prestressed Reinforcement (Required near top of girder)

The following method is used to calculate the non-prestressed reinforcement in the top flange at the end of the girder. **LRFD [T-5.9.4.1.2-1]**



$\boxed{f_{tt} = -0.303}$ ksi

$\boxed{f_{bt} = 1.896}$ ksi

$\boxed{D_s = 1.75}$ feet

$\boxed{b = 48}$ inches

$$Y := \frac{f_{tt} \cdot D_s \cdot 12}{f_{tt} - f_{bt}}$$ $\boxed{Y = 2.898}$ inches

$$T := \frac{|f_{tt}| \cdot b \cdot Y}{2}$$ $\boxed{T = 21.101}$ kips

$\boxed{f_y = 60}$

$$A_{reqd} := \frac{T}{0.5 \cdot f_y}$$ $\boxed{A_{reqd} = 0.703}$ in^2

Therefore, use standard reinforcement; 5 #4 bars, As = 5*0.20 = 1.00 in^2

E19-3.14 Longitudinal Tension Flange Capacity:

The total capacity of the tension reinforcing must meet the requirements of **LRFD [5.8.3.5].** The capacity is checked at the critical section for shear:

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\left| \frac{V_{u\_crit}}{\phi_v} - V_{p\_cw} \right| - 0.5 \cdot V_s \right) \cdot \cot\theta \quad \boxed{T_{ps} = 241} \text{ kips}$$

actual capacity of the straight strands:

$$\boxed{N \cdot A_s \cdot f_{pu\_crit} = 354} \text{ kips}$$

Is the capacity of the straight strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

Check the tension capacity at the edge of the bearing:

The strand is anchored $l_{px} := 8$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.11.4.2]**:

$$\boxed{l_{tr} = 30.00} \text{ in}$$

$$\boxed{l_d = 70.0} \text{ in}$$

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$$Y_s := |y_b - e_s| \quad \boxed{Y_s = 2.25} \text{ in}$$

$$l_{px'} := l_{px} + Y_s \cdot \cot\theta \quad \boxed{l_{px'} = 12.05} \text{ in}$$

$$f_{pb} := \frac{f_{pe} \cdot l_{px'}}{60 \cdot d_s} \quad \boxed{f_{pb} = 69.07} \text{ ksi}$$

Tendon capacity of the straight strands: $\boxed{N \cdot A_s \cdot f_{pb} = 169}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$s_{ave} := s$ $\boxed{s_{ave} = 7.00}$ in

$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s_{ave}}$ $\boxed{V_s = 110}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 0}$ kips

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 133}$ kips

Minimum capacity required at the front of the bearing:

$T_{breqd} := \left(\frac{V_{u\_crit}}{\phi_v} - 0.5 \cdot V_s - V_{p\_cw}\right) \cdot \cot\theta$ $\boxed{T_{breqd} = 166}$ kips

Is the capacity of the straight strands greater than T_{breqd}? $\boxed{check = \text{"OK"}}$

E19-3.15 Live Load Deflection Calculations

Check the Live Load deflection with the vehicle loading as specified in **LRFD [3.6.1.3.2]**; design truck alone or 25% of the design truck + the lane load.

The deflection shall be limited to L/800.

The moment of inertia of the entire bridge shall be used.

$\Delta_{limit} := \frac{L \cdot 12}{800}$ $\boxed{\Delta_{limit} = 0.660}$ inches

$\boxed{I = 32942}$ in^4

$\boxed{n_{beams} = 8}$

$I_{bridge} := I \cdot n_{beams}$ $\boxed{I_{bridge} = 263536}$ in^4

From CBA analysis with 2 lanes loaded, the truck deflection controlled:

$\Delta_{truck} := 0.347$ in

Applying the multiple presence factor from **LRFD [Table 3.6.1.1.2-1]** for 2 lanes loaded:

$\Delta := 1.0 \cdot \Delta_{truck}$ $\boxed{\Delta = 0.347}$ in

Is the actual deflection less than the allowable limit, $\Delta < \Delta$ limit? $\boxed{check = \text{"OK"}}$

E19-3.16 Camber Calculations

Moment due to straight strands:

Number of straight strands: $\boxed{N = 16}$

Eccentricity of the straight strands: $\boxed{e_s = -8.25}$ in

$P_{i\_s} := N \cdot A_s \cdot (f_{tr} - \Delta f_{pES})$ $\boxed{P_{i\_s} = 474}$ kips

$M_1 := P_{i\_s} \cdot |e_s|$ $\boxed{M_1 = 3908}$ kip-in

Upward deflection due to straight strands:

Length of the girder: $\boxed{L_g = 45}$ ft

Modulus of Elasticity of the girder at release: $\boxed{E_{ct} = 3952}$ ksi

Moment of inertia of the girder: $\boxed{I = 32942}$ in^4

$$\Delta_s := \frac{M_1 \cdot L_g^2}{8 \cdot E_{ct} \cdot I} \cdot 12^2$$ $\boxed{\Delta_s = 1.07}$ in

Total upward deflection due to prestress:

$\Delta_{PS} := \Delta_s$ $\boxed{\Delta_{PS} = 1.07}$ in

Downward deflection due to beam self weight at release:

$$\Delta_{gi} := \frac{5 \cdot (w_g + w_d) \cdot L_g^4}{384 \cdot E_{ct} \cdot I} \cdot 12^3$$ $\boxed{\Delta_{gi} = 0.44}$ in

Anticipated prestress camber at release:

$\Delta_i := \Delta_{PS} - \Delta_{gi}$ $\boxed{\Delta_i = 0.63}$ in

The downward deflection due to the dead load of the joint grout, overlay, railing and future wearing surface:

Calculate the additional non-composite dead loads for an exterior girder:

$w_{nc} := w_{j\_ext} + w_{o\_ext} + w_{r\_ext} + w_{fws\_ext}$ $\boxed{w_{nc} = 0.241}$ klf

Modulus of Elasticity of the beam at final strength $\boxed{E_B = 5021}$ ksi

$$\Delta_{nc} := \frac{5 \cdot w_{nc} \cdot L^4}{384 \cdot E_B \cdot I} \cdot 12^3$$ $\boxed{\Delta_{nc} = 0.123}$ in

The residual camber for an exterior girder:

$$RC := \Delta_i - \Delta_{nc}$$ $\boxed{RC = 0.507}$ in

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E19-4 Lifting Check for Prestressed Girders, LRFD

This example shows design calculations for the lifting check for the girder in design example E19-1. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

E19-4.1 Design Criteria

$L_{girder} := 146$ feet

$f'_{ci} := 6.8$ ksi

$f_y := 60$ ksi

girder_size = "72W-inch"

$w_{top\_flg} = 48$ inches

$w_{girder} = 0.953$ kips/ft

$t_{top\_flg\_min} = 3$ inches

$S_{bot} = -18825$ in^3

$t_{top\_flg\_max} = 5.5$ inches

$S_{top} = 17680$ in^3

$t_w = 6.5$ inches

Lift point is assumed to be at the 1/10th point of the girder length.

E19-4.2 Lifting Stresses

Initial Girder Stresses (Taken from Prestressed Girder Output):

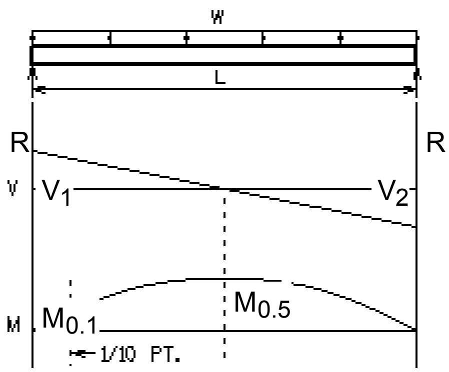
At the 1/10th Point, (positive values indicate compression)

$f_{i\_top\_0.1} := 0.284$ ksi

$f_{i\_bot\_0.1} := 3.479$ ksi

The initial stresses in the girder (listed above) are due to the prestressed strands and girder dead load moment. The girder dead load moment and resulting stresses are based on the girder being simply supported at the girder ends. These resulting stresses are subtracted from the total initial stresses to give the stresses resulting from the pressing force alone.

Moments and Shears due to the girder self weight:



$R := \frac{1}{2} \cdot (w_{girder}) \cdot L_{girder}$ $\boxed{R = 69.569}$ kips

$V_1 := R$ $\boxed{V_1 = 69.569}$ kips

$V_2 := R$ $\boxed{V_2 = 69.569}$ kips

$$M_{gird0.1} := \frac{(w_{girder}) \cdot (0.1 \cdot L_{girder})}{2} \cdot (L_{girder} - 0.1L_{girder})$$

$\boxed{M_{gird0.1} = 914.14}$ kip-ft

Top of girder stresses due to prestress forces:

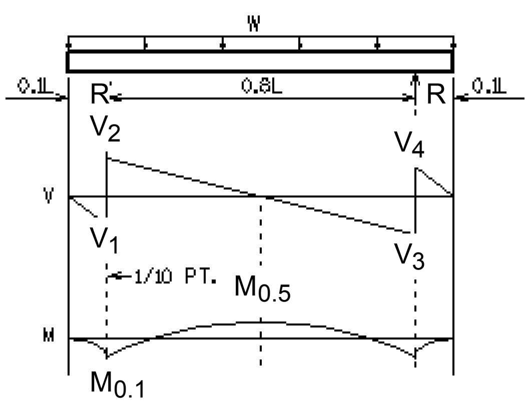
$$f_{top\_prestr} := f_{i\_top\_0.1} - \frac{M_{gird0.1} \cdot 12}{S_{top}}$$ $\boxed{f_{top\_prestr} = -0.336}$ ksi

$$f_{bot\_prestr} := f_{i\_bot\_0.1} - \frac{M_{gird0.1} \cdot 12}{S_{bot}}$$ $\boxed{f_{bot\_prestr} = 4.062}$ ksi

The girder dead load moment and resulting stresses are calculated based on the girder being supported at the lift points. The resulting stresses are added to the stresses due to the prestress forces to give the total stresses during girder picks.

Moments and Shears at the Lift Points, 1/10 point, due to the girder self weight.

$\boxed{R = 69.569}$ kips



$V'_1 := -w_{girder} \cdot 0.1 \cdot L_{girder}$

$\boxed{V'_1 = -13.914}$ kips

$V'_2 := V'_1 + R$ $\boxed{V'_2 = 55.655}$ kips

$V'_3 := V'_2 - (w_{girder} \cdot 0.8 \cdot L_{girder})$

$\boxed{V'_3 = -55.655}$ kips

$V'_4 := V'_3 + R$ $\boxed{V'_4 = 13.914}$ kips

$$M_{gird0.1\_Lift} := \frac{1}{2} \cdot V'_1 \cdot (L_{girder} \cdot 0.1)$$ $\boxed{M_{gird0.1\_Lift} = -101.57}$ kip-ft

Top of girder stresses due to lifting forces (postive stress values indicate compression.):

$$f_{top\_Lift} := f_{top\_prestr} + \frac{M_{gird0.1\_Lift} \cdot 12}{S_{top}}$$ $\boxed{f_{top\_Lift} = -0.405}$ ksi

$$f_{bot\_Lift} := f_{bot\_prestr} + \frac{M_{gird0.1\_Lift} \cdot 12}{S_{bot}}$$ $\boxed{f_{bot\_Lift} = 4.126}$ ksi

E19-4.3 Check Compression Stresses due to Lifting

Check temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

$f_{ciall} := 0.65 \cdot f'_{ci}$ where $f'_{ci} = 6.8$ ksi $\boxed{f_{ciall} = 4.42}$ ksi

Is the stress at the bottom of the girder less than the allowable? $\boxed{check_{f\_bot} = \text{"OK"}}$

If stress at the bottom of girder is greater than allowable, calculate f'_{ci_reqd}

$$f'_{ci\_reqd} := \frac{f_{bot\_Lift}}{0.65}$$

E19-4.4 Check Tension Stresses due to Lifting

The temporary allowable tension, from **LRFD [Table 5.9.4.1.2-1]**, is:

$f_{tall} := -0.24 \cdot \lambda \sqrt{f'_{ci}}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_{tall} = -0.626}$ ksi

$\boxed{f_{top\_Lift} = -0.405}$ ksi

Is the stress at the top of the girder less than the allowable? $\boxed{check_{f\_top} = \text{"OK"}}$

Therefore, proportion the reinforcement in the top flange using an allowable stress of:

$f_s := \min(0.5 \cdot f_y, 30)$ $\boxed{f_s = 30}$ ksi

E19-4.5 Design Top Flange Reinforcement

Calculate the location of the neutral axis:

$\boxed{h_{girder} = 72}$ in

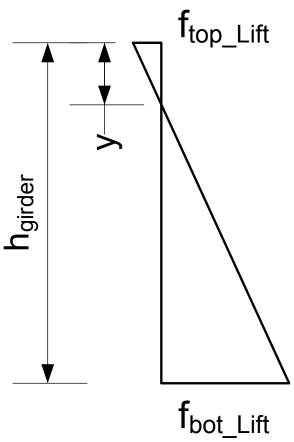
$\boxed{f_{top\_Lift} = -0.405}$ ksi

$\boxed{f_{bot\_Lift} = 4.126}$ ksi

$$y = h_{girder} \cdot \frac{f_{top\_Lift}}{f_{top\_Lift} - f_{bot\_Lift}}$$

$\boxed{y = 6.441}$ in

$\boxed{y_{Location} = \text{"Y is located in the girder web."}}$



Calculate the average flange thickness:

$$A_1 = \frac{1}{2} \cdot (t_{top\_flg\_min} + t_{top\_flg\_max}) \cdot (w_{top\_flg} - t_w) \qquad \boxed{A_1 = 176.375} \quad in^2$$

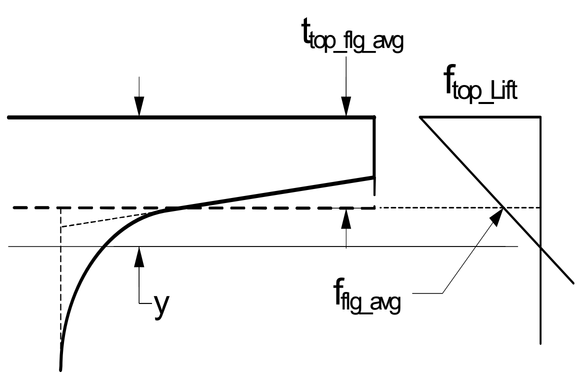
$$t_1 = \frac{1}{2} \cdot (t_{top\_flg\_min} + t_{top\_flg\_max}) \qquad \boxed{t_1 = 4.25} \quad in$$

$$A_2 = t_{top\_flg\_max} \cdot t_w \qquad \boxed{A_2 = 35.75} \quad in^2$$

$$t_2 = t_{top\_flg\_max} \qquad \boxed{t_2 = 5.5} \quad in$$

$$t_{top\_flg\_avg} = \frac{A_1 \cdot t_1 + A_2 \cdot t_2}{A_1 + A_2} \qquad \boxed{t_{top\_flg\_avg} = 4.461} \quad in$$

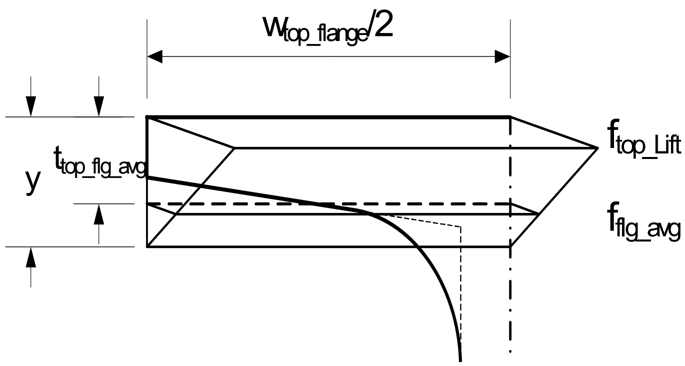
Determine the values of the stress at the average flange thickness.



At $\boxed{t_{top\_flg\_avg} = 4.461}$ inches from the top of the girder:

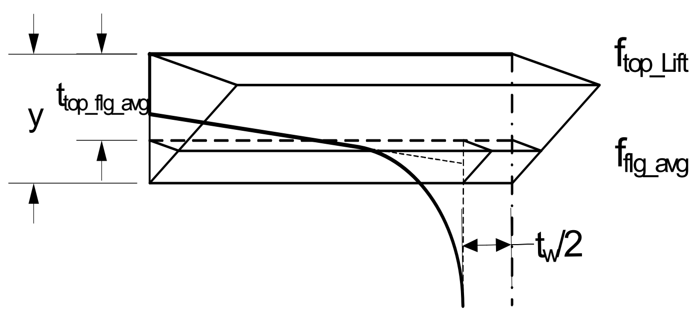
$$f_{flg\_avg} = \frac{f_{top\_Lift}}{y} \cdot \left(y - t_{top\_flg\_avg}\right) \qquad \boxed{f_{flg\_avg} = -0.125} \quad \text{ksi}$$

Calculate the tension force in the girder flange.



$$T_{flg\_avg} = \frac{1}{2} \cdot \left(f_{top\_Lift} + f_{flg\_avg}\right) \cdot t_{top\_flg\_avg} \cdot w_{top\_flg} \qquad \boxed{T_{flg\_avg} = -56.742} \quad \text{kips}$$

Calculate the tension force in the girder web (this minor force can be ignored for simplification).



$$T_{web} = \frac{1}{2} \cdot f_{flg\_avg} \cdot \left(y - t_{top\_flg\_avg}\right) \cdot t_w \qquad \boxed{T_{web} = -0.802} \quad \text{kips}$$

$$T_{total} = T_{flg\_avg} + T_{web} \qquad \boxed{T_{total} = -57.544} \quad \text{kips}$$

$$\boxed{T = 57.544} \quad \text{kips}$$

$$As_{Reqd} := \frac{T}{f_s} \qquad \boxed{As_{Reqd} = 1.918} \quad \text{in}^2$$

Use 6 bars in the Top Flange: $Number\_Bars := 6$

Try #6 Bars: $Bar_{No} \equiv 6$

$A_s := \frac{As_{Reqd}}{Number\_Bars}$ $\boxed{A_s = 0.32}$ in^2 per bar

Area of a #6 Bar: $\boxed{Bar_A(Bar_{No}) = 0.442}$ in^2 per bar

Is the area of steel per bar greater than required? $\boxed{check_{As} = \text{"OK"}}$

Therefore, use 6 - #6 Bars in Top Flange of Girder for 0.1 point lifting locations.

Note that these bars should be terminated where no longer required by design and lapped with 6 #4 bars as shown on the Standard Details.

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Table of Contents

23.1 Introduction

23.1.1 General

This chapter covers the design of spike “laminated deck” superstructures made of timber. This type of structure has a laminated wood deck, where a series of laminations are placed edgewise and oriented in the direction of the span of the bridge. They are spiked together on their wide face with deformed spikes to create a laminated deck panel. These deck panels are prefabricated at a plant in panels less than 7'-6" wide, so they can be easily shipped to the bridge site. At the bridge site the panels are joined together by driving spikes through the ship-lap joint. To assist in spreading applied loads transversely across the deck, stiffener beams are provided. These beams are attached to the underside of each deck panel near its edges and at intermediate points. The timber deck members are treated with a preservative prior to shipping. This will protect the timber against decay and insects, and it will also retard weathering and checking. A bituminous overlay or wearing surface is placed on top of the deck to provide a good riding surface and to protect the deck.

Other types of timber bridges not discussed in this chapter include timber trusses, arches, box culverts, girders, glu-laminated girders and parallel chord timber bridges.

The spike “laminated deck” is one of the least complex bridge types to construct. It is composed of simple spans between each support. It has a superstructure composed of a single material which is easy to fabricate and install. Its limitation lies in the practical range of span lengths for its application.

Timber bridges are aesthetically pleasing and blend well in natural surroundings. These bridges can be constructed in any weather, including cold and wet conditions, without detrimental effects. They are resistant to the effects of deicing agents. The lighter weight of timber allows for easier fabrication and construction since smaller equipment can be used to lift the members into place. Timber bridges tend to deteriorate faster if subjected to high repetitions of heavy loads. Their cost effectiveness should also be evaluated for each site.

23.1.2 Limitations

Timber bridges are not recommended over streams where the 100 year (Q_{100}) frequency flood discharge provides a freeboard less than 24 inches.

They are also not recommended on highways where the Average Daily Traffic (ADT) is greater than 400 vehicles per day The Average Daily Truck Traffic (ADTT) should be significantly less than 100 trucks per day before these timber bridges are allowed **LRFD [9.9.6.1]**.

23.2 Specifications, Material Properties and Deck Thickness

23.2.1 Specifications

Reference may be made to the design and construction related material as presented in the following specifications:

- *State of Wisconsin, Department of Transportation Standard Specifications for Highway and Structure Construction*

  Section 507 Timber Structures

- Other Specifications as referenced in Chapter 3
- *National Design Specifications for Wood Construction (NDS)*
- *American Institute of Timber Construction (Manual) - (AITC)*

23.2.2 Material Properties

23.2.2.1 Reference Design Values

The reference design values for timber members used in laminated deck panels are defined as follows:

Douglas Fir – Larch (No. 1 & Better) – Visually Graded Sawn Lumber **LRFD [Table 8.4.1.1.4-1]**

F_{bo} = 1.20 ksi = reference design value in bending (flexure)

F_{vo} = 0.180 ksi = reference design value in shear

F_{cpo} = 0.625 ksi = reference design value in compression perpendicular to grain

E_o = 1800 ksi = reference modulus of elasticity

Reference design values are based on dry-use conditions, with the wood moisture content not exceeding 19 percent for sawn lumber **LRFD [C8.4.1]**. Reference design values also apply to material treated with preservatives in accordance with *AASHTO Standard Specifications for Transportation Materials* M133 **LRFD [8.4.1]**.

23.2.2.2 Adjusted Design Values

Adjusted design values shall be obtained by multiplying the reference design values by applicable adjustment factors in accordance with **LRFD [8.4.4]** and shown below. All units are in ksi.

$F_b = F_{bo}\ C_{KF}\ C_M\ (\ C_F \text{ or } C_v\ )\ C_{fu}\ C_i\ C_d\ C_\lambda$ = adjusted design value in bending (flexure)

$F_v = F_{vo} C_{KF} C_M C_i C_\lambda$ = adjusted design value in shear

$F_{cp} = F_{cpo} C_{KF} C_M C_i C_\lambda$ = adjusted design value in compression perpendicular to grain

$E = E_o C_M C_i$ = adjusted modulus of elasticity

Where:

C_{KF}	=	Format conversion factor specified in **LRFD [8.4.4.2]**
C_M	=	Wet service factor specified in **LRFD [8.4.4.3]**
C_F	=	Size factor for visually graded dimension lumber and sawn timbers specified in **LRFD [8.4.4.4]**
C_v	=	Volume factor for structural glued laminated timber specified in **LRFD [8.4.4.5]**
C_{fu}	=	Flat use factor specified in **LRFD [8.4.4.6]**
C_i	=	Incising factor specified in **LRFD [8.4.4.7]**
C_d	=	Deck factor specified in **LRFD [8.4.4.8]**
C_λ	=	Time effect factor specified in **LRFD [8.4.4.9]**

23.2.2.2.1 Format Conversion Factor, C_{KF}

The reference design value is multiplied by the format conversion factor, C_{KF} , to go from a value that is used in allowable stress design to a value that is used with load and resistance factor design **LRFD [8.4.4.2]**. Use a C_{KF} value of $2.5/\phi$, except for compression perpendicular to the grain which shall use a value of $2.1/\phi$. The resistance factors, ϕ , are provided in **LRFD [8.5.2.2]**.

23.2.2.2.2 Wet Service Factor, C_M

The reference design value is based on dry use resistance and shall be modified for moisture content using the wet service factor, C_M. For sawn lumber with an in-service moisture content of 19% or less, use a C_M value of 1.0. Otherwise, see **LRFD [8.4.4.3]**.

23.2.2.2.3 Size Factor for Sawn Lumber, C_F

The size factor, C_F , shall have a value of 1.0, unless otherwise specified by **LRFD [Table 8.4.4.4-1]**.

23.2.2.2.4 Volume Factor, C_v , (Glulam)

The volume factor, C_v , doesn't apply to laminated deck structures, but to horizontally laminated glulam members **LRFD [8.4.4.5]**.

23.2.2.2.5 Flat Use Factor, C_{fu}

The flat use factor, C_{fu} , doesn't apply to laminated deck structures, but to specific grades of planks with load applied to the wide face and vertically laminated glulam with loads applied parallel to the wide face of laminations **LRFD [8.4.4.6]**.

23.2.2.2.6 Incising Factor, C_i

The reference design values for dimension lumber shall be multiplied by the incising factor specified in **LRFD [Table 8.4.4.7-1]** when members are incised parallel to the grain a maximum depth of 0.4 inches, a maximum length of 3/8 inches, and a density of incisions up to 1100/ft^2.

23.2.2.2.7 Deck Factor, C_d

For spike "laminated decks" constructed of solid sawn lumber 2 to 4 inches thick, F_{bo} may be adjusted by multiplying it by C_d as specified in **LRFD [Table 8.4.4.8-1]**. Laminated decks exhibit an increased resistance in bending. The value for C_d in this table is 1.15.

23.2.2.2.8 Time Effect Factor, C_λ

The time effect factor, C_λ , shall be chosen to respond to the appropriate strength limit state as specified in **LRFD [Table 8.4.4.9-1]**. For Strength I Limit State the value for C_λ is 0.8.

23.2.3 Deck Thickness

Deck Thickness (inches)	Effective Span (L) [1] (feet)
10	L = 17
12	17 < L ≤ 25
14	25 < L ≤ 30
16	30 < L ≤ 36

Table 23.2-1
Deck Thickness vs. Effective Span

[1] The effective span shall be taken as the clear distance between supports plus one half the width of one support, but not to exceed the clear span plus the deck thickness.

23.3 Limit States Design Method

23.3.1 Design and Rating Requirements

All new laminated deck structures are to meet design requirements as stated in 17.1.1 and rating requirements as stated in 17.1.2.

23.3.2 LRFD Requirements

23.3.2.1 General

For laminated deck design, the deck dimensions, length of bearing at support and the spacing of spikes at the ship-lap joint shall be selected to satisfy the equation below for all appropriate Limit States: **LRFD [1.3.2.1]**

$$Q = \sum \eta_i \, \gamma_i \, Q_i \leq \phi \, R_n = R_r$$ (Limit States Equation) **LRFD [1.3.2.1, 3.4.1]**

Where:

η_i	=	Load modifier (a function of η_D , η_R and η_I) **LRFD [1.3.2.1, 1.3.3, 1.3.4, 1.3.5]**
γ_i	=	Load factor
Q_i	=	Force effect; moment, shear or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance; resistance of a component to force effects
R_r	=	Factored resistance = $\phi \, R_n$

The Limit States used for laminated deck design are:

- Strength I Limit State
- Service I Limit State

23.3.2.2 Statewide Policy

Current Bureau of Structures policy is :

- Set value of load modifier, η_i , and its factors (η_D , η_R , η_I) all equal to 1.00 for laminated deck design.

- Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects, Q_i , for laminated deck bridges.

- Ignore reduction factor, r, for skewed laminated deck bridges in **LRFD [4.6.2.3]** that would reduce longitudinal force effects, Q_i .

23.3.3 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a bridge is expected to experience during its design life **LRFD [1.3.2.4]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 23.3.2.1.

Strength I Limit State **LRFD [3.4.1]** will be used for:

- Designing laminated deck for bending (flexure)

- Checking horizontal shear in laminated deck near the supports

- Checking compression perpendicular to grain in laminated deck at the supports

- Checking spacing of drive spikes at ship-lap joint

23.3.3.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 23.3.2.2.

Strength I Limit State will be used to design the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 23.4.3 and appropriate (HL-93) live loads, LL and IM, defined in 23.4.4.1. When sidewalks are present, include force effects of pedestrian live load, PL, defined in 23.4.4.2.

The load factor, γ_i , is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

For Strength I Limit State, the values of γ_i for each applied load, are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]** and their values are: γ_{DC} = 1.25/0.90, γ_{DW} = 1.50/0.65, γ_{LL+IM} = γ_{PL} = 1.75. The values for γ_{DC} and γ_{DW} have a maximum and minimum value.

Therefore, for Strength I Limit State:

$$Q = 1.0 \, [\, 1.25(DC) + 1.50(DW) + 1.75((LL + IM) + PL) \,]$$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads. The load factors shown for DC and DW are maximum values. Use maximum or minimum values as shown in **LRFD [Table 3.4.1-2]** to calculate the critical force effect.

23.3.3.2 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for Strength Limit State **LRFD [8.5.2.2]** are:

- ϕ = 0.85 for flexure
- ϕ = 0.75 for shear
- ϕ = 0.90 for compression perpendicular to grain
- ϕ = 0.65 for connections

The factored resistance, R_r (M_r , V_r , P_r), associated with the list of items to be designed/checked using Strength I Limit State in 23.3.3, are described in the following sections.

23.3.3.2.1 Moment Capacity

For rectangular sections, the nominal moment resistance, M_n , equals: **LRFD [8.6.2]**

$$M_n = F_b\ S\ C_L$$

Where:

F_b	=	Adjusted design value in bending (flexure) specified in **LRFD [8.4.4.1]** (ksi)
S	=	Section modulus = $b\ d^2 / 6$ (in^3)
b	=	Net width, as specified in **LRFD [8.4.1.1.2]** (in)
d	=	Net depth, as specified in **LRFD [8.4.1.1.2]** (in)
C_L	=	Beam stability factor

The factored resistance, M_r , or moment capacity, shall be taken as: **LRFD [8.6.1]**

$$M_r = \phi\ M_n = \phi\ F_b\ S\ C_L$$

For timber members in flexure, the resistance factor, ϕ, is 0.85 and for spike “laminated decks” the value for C_L is 1.0, therefore:

$$M_r = (0.85)\ F_b\ S$$

23.3.3.2.2 Shear Capacity

The nominal shear resistance, V_n , shall be determined as: **LRFD [8.7]**

$$V_n = F_v \, b \, d / 1.5 \quad \text{(kips)}$$

Where:

F_v	=	Adjusted design value in shear, specified in **LRFD [8.4.4.1]** (ksi)
b	=	Net width, as specified in **LRFD [8.4.1.1.2]** (in)
d	=	Net depth, as specified in **LRFD [8.4.1.1.2]** (in)

The factored resistance, V_r , or shear capacity of a component of rectangular cross-section, shall be taken as: **LRFD [8.7]**

$$V_r = \phi \, V_n = \phi \, F_v \, b \, d / 1.5$$

The resistance factor for shear, ϕ, is 0.75, therefore:

$$V_r = (0.75) \, F_v \, b \, d / 1.5$$

23.3.3.2.3 Compression Perpendicular to Grain Capacity

The nominal resistance, P_n , of a member in compression perpendicular to grain shall be taken as: **LRFD[8.8.3]**

$$P_n = F_{cp} \, A_b \, C_b$$

Where:

F_{cp}	=	Adjusted design value in compression perpendicular to grain as specified in **LRFD [8.4.4.1]** (ksi)
A_b	=	Bearing area (in^2)
C_b	=	Bearing adjustment factor as specified in **LRFD [Table 8.8.3-1]**

When the bearing area is in a location of high flexural stress or is closer than 3 inches from the end of the component, C_b , shall be taken as 1.0. In all other cases, C_b , shall be as specified in **LRFD [Table 8.8.3-1]**.

The factored resistance, P_r , or compression capacity, shall be taken as: **LRFD [8.8.1]**

$$P_r = \phi \, P_n = \phi \, F_{cp} \, A_b \, C_b$$

For compression perpendicular to grain, the resistance factor, ϕ, is 0.90, therefore:

$P_r = (0.9)\ F_{cp}\ A_b\ C_b$

23.3.4 Service Limit State

Service I Limit State shall be applied as a restriction on deformation under regular service conditions **LRFD [1.3.2.2]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 23.3.2.1.

Service I Limit State **LRFD [3.4.1]** will be used for:

- Checking live load deflection criteria

23.3.4.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 23.3.2.2.

Service I Limit State will be used to analyze the structure for force effects, Q_i , due to appropriate (HL-93) live loads, LL and IM, defined in 23.4.4.1.

For Service I Limit State, the value of γ_i for applied live load, is found in **LRFD [Table 3.4.1-1]** and its value is: $\gamma_{LL+IM} = 1.0$

Therefore, for Service I Limit State:

$$Q = 1.0\ [\ 1.0(LL + IM)\ ]$$

Where LL and IM represent force effects due to these applied loads.

23.3.4.2 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

The factored resistance, R_r , associated with the checking of live load deflection using Service I Limit State is described below.

23.3.4.2.1 Live Load Deflection Criteria

All spike “laminated deck” structures shall be designed to meet live load deflection limits. Large deflections in wood components can cause fasteners to loosen and wearing surfaces to deteriorate. The limit for live load deflections for laminated deck structures is L/425 for vehicular and pedestrian loads **LRFD [2.5.2.6.2]**. The deflections are based on entire deck width acting as a unit and net-section moment of inertia, I_{net} .

The nominal resistance, R_n , or deflection limit, is:

$$R_n = L/425$$

Where:

L = Span length

The factored resistance, R_r , is:

$R_r = \phi\ R_n = \phi\ (L/425)$

The resistance factor, ϕ, is 1.00, therefore:

$R_r = (1.0)\ R_n = (L/425)$

23.3.5 Fatigue Limit State

Fatigue need not be investigated for wood decks (laminated decks) as described in **LRFD [9.5.3, 9.9]**

23.4 Laminated Deck Design Procedure

23.4.1 Trial Deck Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, skew, roadway width, etc.. Knowing the span lengths, a trial deck depth can be obtained from Table 23.2-1.

NOTE: With preliminary structure sizing complete, check to see if structure exceeds limitations in 23.1.2.

23.4.2 Dimensions

Structural calculations shall be based on the actual net dimensions for the anticipated use conditions. These net dimensions depend on the type of surfacing used on the timber member. See **LRFD [8.4.1.1.2]** for a description of dimensions to use.

23.4.3 Dead Loads (DC, DW)

Dead loads (permanent loads) are defined in **LRFD [3.3.2]**. Timber dead load is computed by using a unit weight of 50 pcf **LRFD [3.5.1]**. This value includes the weight of mandatory preservatives used to treat the wood. The bituminous wearing surface load is computed by using a unit weight of 150 pcf.

DC	=	dead load of structural components and any nonstructural attachments
DW	=	dead load of bituminous wearing surface, future wearing surface (F.W.S.) and utilities

The laminated deck dead load, DC_{deck} , and the bituminous wearing surface load, DW_{bitws} , are included in the design. A post dead load, DW_{FWS} , of 20 psf, for possible future wearing surface (F.W.S.), is required in the design by the Bureau of Structures.

Dead loads, DC, from railings, curbs and scupper blocks are uniformly distributed across the full width of the deck when designing an interior strip. For the design of exterior strips, any of these dead loads, DC, that are located directly over the exterior strip width shall be applied to the exterior strip. For both interior and exterior strips, the future wearing surface, DW_{FWS} , and bituminous wearing surface, DW_{bitws} , located directly over the strip width shall be applied to it.

23.4.4 Live Loads

23.4.4.1 Vehicular Live Load (LL) and Dynamic Load Allowance (IM)

The *AASHTO LRFD Specifications* contain several live load components (see 17.2.4.2) that are combined and scaled to create live load combinations that apply to different Limit States **LRFD [3.6.1]**. Where the equivalent strip method is used as described in 23.4.6, and the span exceeds 15 feet, all of the live loads specified in **LRFD [3.6.1.2]** shall be applied **LRFD**

[3.6.1.3.3]. Live load combinations (LL#3 and LL#4) as shown in 17.2.4.2.6, do not apply because all spans in laminated deck structures are simple spans and the Fatigue Limit State does not apply to laminated decks.

The live load combinations used for design are:

LL#1:	Design Tandem (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#2:	Design Truck (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#5:	Design Truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6:	25% [Design Truck (+ IM)] + Design Lane Load	**LRFD [3.6.1.3.2]**

Table 23.4-1
Live Load Combinations

The dynamic load allowance, IM, **LRFD [3.6.2.3]** need not be applied to wood components. Wood structures are known to experience reduced dynamic wheel load effects due to internal friction between the components and the damping characteristics of wood. Additionally, wood is stronger for short duration loads, as compared to longer duration loads. This increase in strength is greater than the increase in force effects resulting from the dynamic load allowance.

The live load combinations are applied to the Limit States as shown in Table 23.4-2.

The live load force effect, Q_i , shall be taken as the largest from the live loads shown in Table 23.4-2 for that Limit State.

Strength I Limit State: [1]	LL#1 , LL#2	IM = 0%
Service I Limit State: (for LL deflection criteria)	LL#5 , LL#6	IM = 0%

Table 23.4-2
Live Loads for Limit States

[1] Load combinations shown are used for design of interior strips and exterior strips.

23.4.4.2 Pedestrian Live Load (PL)

For bridges designed for both vehicular and pedestrian live load, a pedestrian live load, PL, of 75 psf is used. However, for bridges designed exclusively for pedestrian and/or bicycle traffic, a live load of 85 psf is used **LRFD [3.6.1.6]**. The dynamic load allowance, IM, is not applied to pedestrian live loads **LRFD [3.6.2]**.

Pedestrian loads are not applied to an interior strip for its design. For the design of exterior strips, any pedestrian loads that are located directly over the exterior strip width shall be applied to the exterior strip.

23.4.5 Minimum Deck Thickness Criteria

Check adequacy of chosen deck thickness by looking at live load deflection criteria, using Service I Limit State.

23.4.5.1 Live Load Deflection Criteria

All laminated deck structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2, 9.9.3.3]**. Live load deflections for laminated deck structures are limited to L/425. The live load deflection, Δ_{LL+IM} , shall be calculated using factored loads described in 23.3.4.1 and 23.4.4.1 for Service I Limit State.

Place live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**. Use net-section moment of inertia, I_{net} , based on entire deck width acting as a unit. Use adjusted modulus of elasticity, E as described in 23.2.2.2. The factored resistance, R_r , is described in 23.3.4.2.1.

Then check that, $\Delta_{LL+IM} \leq R_r$ is satisfied.

23.4.6 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below. The equivalent distribution width applies for both live load moment and shear.

23.4.6.1 Interior Strip

Equivalent interior strip widths for laminated deck bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3]** for spans more than 15 feet.

The live loads to be placed on these widths are <u>axle loads</u> (i.e., two lines of wheels) and the <u>full lane load</u>.

Single-Lane Loading: $E = 10.0 + 5.0\ (L_1\ W_1)^{1/2}$

Multi-Lane Loading: $E = 84.0 + 1.44(L_1\ W_1)^{1/2} \leq 12.0(W)/N_L$

Where:

E = Equivalent distribution width (in)

L_1 = Modified span length taken equal to the lesser of the actual span or 60.0 ft (ft)

W_1 = Modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60.0 ft for multi-lane loading, or 30.0 ft for single-lane loading (ft)

W = Physical edge to edge width of bridge (ft)

N_L = Number of design lanes as specified in **LRFD [3.6.1.1.1]**

23.4.6.1.1 Strength Limit State

Use the smaller equivalent width (single-lane or multi-lane), when (HL-93) live load is to be distributed for Strength I Limit State.

The distribution factor, DF, is computed for a design deck width equal to one foot.

$$DF = \frac{1}{E}$$

Where:

E = Equivalent distribution width (ft)

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore aren't used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

23.4.6.2 Exterior Strip

Equivalent exterior strip widths for laminated deck bridges are covered in **LRFD [4.6.2.1.4]**.

The exterior strip width, E, is assumed to carry one wheel line and a tributary portion of design lane load (located directly over the strip width).

E equals the distance between the edge of the deck and the inside face of the barrier, plus 12 inches, plus ¼ of the full strip width specified in **LRFD [4.6.2.3]**.

The exterior strip width, E, shall not exceed either ½ the full strip width or 72 inches.

Use the smaller equivalent width (single-lane or multi-lane), for full strip width, when (HL-93) live load is to be distributed for Strength I Limit State.

The multiple presence factor, m, has been included in the equations for full strip width and therefore aren't used to adjust the distribution factor **LRFD [3.6.1.1.2]**.

23.4.6.2.1 Strength Limit State

The distribution factor, DF, is computed for a design deck width equal to one foot.

Compute the distribution factor associated with one truck wheel line, to be applied to <u>axle loads</u>:

$$DF = \frac{(1 \text{ wheel line})}{(2 \text{ wheel lines/lane})(E)}$$

Where:

E = Equivalent distribution width (ft)

Compute the distribution factor associated with tributary portion of design lane load, to be applied to full lane load: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\frac{(SWL)}{(10 \text{ ft lane load width})}\right]}{(E)}$$

Where:

E = Equivalent distribution width (ft)

SWL = Deck width loaded (ft)

= E – (distance from edge of deck to inside face of barrier or curb) (ft)

23.4.7 Design Deck for Strength in Bending

The total factored moment, M_u , shall be calculated using factored loads described in 23.3.3.1 for Strength I Limit State.

The factored resistance, M_r , or moment capacity, shall be calculated as in 23.3.3.2.1.

Then check that, $M_u \le M_r$ is satisfied.

The laminated deck should also be checked for moment capacity (factored resistance), to make sure it can handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1.

23.4.8 Check for Shear

Shear shall be investigated at a distance away from the face of support equal to the depth of the component. When calculating the maximum design shear, the live load shall be placed so as to produce the maximum shear at a distance from the support equal to the lesser of either three times the depth, d , of the component or one-quarter of the span L.

The critical section is between one and three depths from the support.

The critical shear in flexural components is horizontal shear acting parallel to the grain of the component. The resistance of bending components in shear perpendicular to grain need not be investigated.

The factored shear, V_u , shall be calculated using factored loads described in 23.3.3.1 for Strength I Limit State.

The factored resistance, V_r , or shear capacity, shall be calculated as in 23.3.3.2.2.

Then check that, $V_u \leq V_r$ is satisfied.

The laminated deck should have the shear capacity to handle the dead loads and Permit Vehicle as discussed in 23.4.7.

23.4.9 Check Compression Perpendicular to Grain

The factored compression perpendicular to the grain, P_u , shall be calculated using factored loads described in 23.3.3.1 for Strength I Limit State.

The factored resistance, P_r , or compression capacity, shall be calculated as in 23.3.3.2.3.

Then check that, $P_u \leq P_r$ is satisfied.

The laminated deck should have the compression capacity to handle the dead loads and Permit Vehicle as discussed in 23.4.7.

23.4.10 Check Spacing of Drive Spikes at Ship-Lap Joint

Check the spacing of drive spikes at the ship-lap joint to make sure it is adequate to provide sufficient capacity to resist the factored horizontal shear forces along the length of the span.

23.4.11 Fabrication of Deck Panels

The laminations in deck panels are spiked together on their wide faces with deformed spikes of sufficient length to fully penetrate four laminations. The spikes shall be placed in lead holes that are bored through pairs of laminations at each end and at intervals not greater than 12 inches in an alternating pattern near the top and bottom of the laminations, as shown in **LRFD [9.9.6]**. Laminations shall not be butt spliced within their unsupported length. The typical thickness of the laminations is 4 inches. The deck panels are prefabricated at a plant in panel widths less than 7’-6” wide, so it can easily be shipped to the bridge site. The specified design details for lamination arrangement and spiking are based upon current practice. It is important that the spike lead holes provide a tight fit to ensure proper load transfer between laminations and to minimize mechanical movements. See **LRFD [9.9.6]** for spike layout for spike “laminated decks.”

23.4.12 Thermal Expansion

Thermal expansion may be neglected in spike "laminated decks". Generally, thermal expansion has not presented problems in wood deck systems. Most wood decks inherently contain gaps at the butt joints that can absorb thermal movements **LRFD [9.9.3.4]**.

23.4.13 Wearing Surfaces

Laminated decks shall be provided with a wearing surface conforming to the provisions of **LRFD [9.9.8]**. Experience has shown that unprotected wood deck surfaces are vulnerable to wear and abrasion and/or may become slippery when wet.

23.4.14 Deck Tie-Downs

Where deck panels are attached to wood supports, the tie-downs shall consist of metal brackets that are bolted through the deck and attached to the sides of the supporting component. Lag screws or deformed shank spikes may be used to tie panels down to the wood support **LRFD [9.9.4.2]**.

23.4.15 Transverse Stiffener Beam

Interconnection of panels should be made with transverse stiffener beams attached to the underside of the deck. The distance between stiffener beams shall not exceed 8 feet, and the rigidity, EI, of each stiffener beam shall not be less than 80,000 kip-in^2. The beams shall be attached to each deck panel near the panel edges and at intervals not exceeding 15 inches **LRFD [9.9.4.3.1]**.

23.4.16 Metal Fasteners and Hardware

Attachments and fasteners used in wood construction shall be of stainless steel , malleable iron, aluminum or steel that is galvanized, cadmium plated, or otherwise coated to provide durability **LRFD [2.5.2.1.1]**. Material property requirements for metal fasteners and hardware are covered in **LRFD [8.4.2]**. The design of fasteners and connections is covered in **LRFD [8.13]**.

23.4.17 Preservative Treatment

All wood used for permanent applications shall be pressure impregnated with wood preservatives in accordance with the requirements of *AASHTO Standard Specifications for Transportation Materials* M133. Insofar as is practicable, all wood components shall be designed and detailed to be cut, drilled, and otherwise fabricated prior to pressure treatment with wood preservatives. When cutting, boring or other fabrication is necessary after preservative treatment, exposed, untreated wood shall be specified to be treated in accordance with the requirements of *AASHTO* M133. See **LRFD [8.4.3]** for other preservative treatment requirements.

23.4.18 Timber Rail System

Use approved crash-tested rail systems only.

23.4.19 Rating of Superstructure

Refer to *AASHTO LRFR Specification* and also the example that follows.

23.5 References

1. Wipf, T. J., "Load *Distribution Criteria for Glued Laminated Longitudinal Timber Deck Highway Bridges*", Engineering Research Institute, Iowa State University. 1985.

2. Transportation Research Board, "*Timber Bridges*", (Transportation Research Record #1053). 1989.

3. Forest Products Laboratory, "Design*, Fabrication, Testing, and Installation of a Press-Lam Bridges*", (Research Paper FPL-332) 1979.

4. AITC, "*Modern Timber Highway Bridges, A State of the Art Report*", (American Institute of Timber Construction, July 1, 1973).

5. Bohannan, Bill, "*FLP Timber Bridge Deck Research*," Journal of the Structural Division, ASCE, Vol. 98 No. ST3, Proc. Paper 8779, March, 1972, pp. 729-740.

6. Forest Products Laboratory, "*Procedure for Design of Glued-Laminated Orthotropic Bridge Decks*", (Research Paper FPL-210) 1973.

7. Forest Products Laboratory, "*Erection Procedure for Glued-Laminated Timber Bridge Decks with Dowel Connectors*", (Research Paper FPL-263) 1970.

8. Ou, Fong L., Ph.D., P.E., "*An Overview of Timber Bridges*", Engineering Staff Forest Service. (Transportation Research Board Paper-65th Annual Meeting) 1986.

23.6 Design Example

E23-1 Two-Span Timber Bridge, 14 inch Deck, LRFD

(This Design Example will be added in the future)

Table of Contents

24.1 Introduction

Steel girders are recommended due to depth of section considerations for short span structures and due to their economy in comparison with other materials or structure types for longer span structures.

24.1.1 Types of Steel Girder Structures

This chapter considers the following common types of steel girder structures:

- Plate girder
- Rolled girder
- Box girder

A plate girder structure is selected over a rolled girder structure for longer spans or when greater versatility is required. Generally rolled girders are used for web depths less than 36" on short span structures of 80' or less.

24.1.2 Structural Action of Steel Girder Structures

Box girder, rolled girder and plate girder bridges are primarily flexural structures which carry their loads by bending between the supports. The degree of continuity of the steel girders over their intermediate supports determines the structural action within the steel bridge. The main types of structural action are as follows:

- Simply-supported structures
- Multiple-span continuous structures
- Multiple-span continuous hinged structures

Simply-supported structures are generally used for single, short-span structures. Multiple-span steel girder structures are designed as continuous spans. When the overall length of the continuous structure exceeds approximately 900', a transverse expansion joint is provided by employing girder hinges and a modular watertight expansion device.

The 900’ guideline is based on the abutments having expansion bearings and a pier or piers near the center of the continuous segment having fixed bearings. More than one fixed pier shall be used when four or more piers are utilized or when a steep grade (greater than 3%) exists. When one abutment has fixed bearings, see Chapter 12 – Abutments for the limitation on the length of a continuous segment.

24.1.3 Fundamental Concepts of Steel I-Girders

This section describes basic concepts of I-girder sections to aid in understanding the design provisions for steel I-sections presented in *AASHTO LRFD*. This section is cursory in nature.

The behavior of non-composite steel I-section members subject to flexure is similar to the behavior of composite I-section members in negative flexure. A qualitative bending moment versus rotation relationship for a homogeneous compact web section is presented Figure 24.1-1.

A homogeneous section is defined as a section in which the flanges and web have the same nominal yield strength.

In *AASHTO LRFD*, a compact web section is defined as a non-composite section (or a composite section in negative flexure) that has a web with a slenderness at or below which the section can achieve a maximum flexural resistance, M_{max}, equal to the plastic moment, M_p, prior to web bend-buckling having a statistically significant influence on the response. In addition, specific steel grade, ductility, flange slenderness and lateral bracing requirements must also be satisfied. Compact web sections are typically shallower sections, with thicker webs, than non-compact sections. Compact web sections are often rolled beams or welded girder sections with proportions similar to rolled beams.

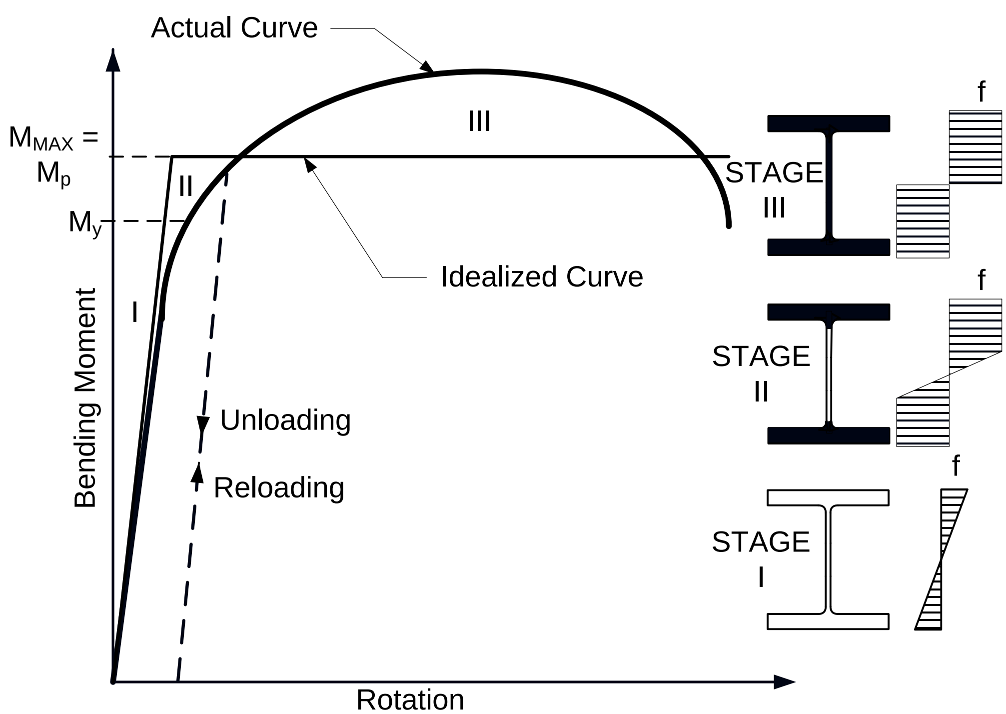

Figure 24.1-1

Bending Moment versus Rotation for Homogeneous Compact Web Section

Proceeding along the actual curve shown in Figure 24.1-1, the initial Stage I behavior represents completely elastic behavior. As the section approaches the theoretical yield moment, M_y, the presence of residual stresses will result in some inelastic behavior in the outer fibers of the cross section before the calculated M_y is reached. At Stage II, yielding continues and begins to progress throughout the section as the section approaches the plastic moment, M_p. At Stage III, the entire cross section has yielded; that is, each component of the cross section is assumed to be at F_y. The idealized curve shown in Figure 24.1-1 is assumed for design. The dotted line shown in Figure 24.1-1 illustrates the behavior of a member that is loaded with a moment greater than M_y and then unloaded.

Figure 24.1-2 shows a moment versus rotation relationship for a homogeneous slender web section. In *AASHTO LRFD*, a slender web section is defined as a non-composite section (or a composite section in negative flexure) that has a web with a slenderness at or above which the theoretical elastic bend-buckling stress in flexure is reached in the web prior to reaching the yield strength of the compression flange. Because web bend-buckling is assumed to occur in such sections, a web load-shedding factor, R_b, must be introduced to account for the effect of the post-bend-buckling resistance or redistribution of the web compressive stresses to the compression flange resulting from the bend-buckling of the web.

The maximum flexural resistance, M_{max}, is taken as the smaller of R_bM_{yc} and M_{yt} for a homogeneous slender-web section, where M_{yc} and M_{yt} are the yield moments with respect to the compression and tension flanges, respectively. Like a compact web section, residual stresses will contribute to yielding and some inelastic behavior will occur prior to reaching M_{max}, as shown in Figure 24.1-2. However, unlike a compact web section, a slender web section has little or no available inelastic rotation capacity after reaching M_{max}. Therefore, the flexural resistance drops off quite rapidly after reaching M_{max}, and redistribution of moments is not permitted when these sections are used at interior piers.

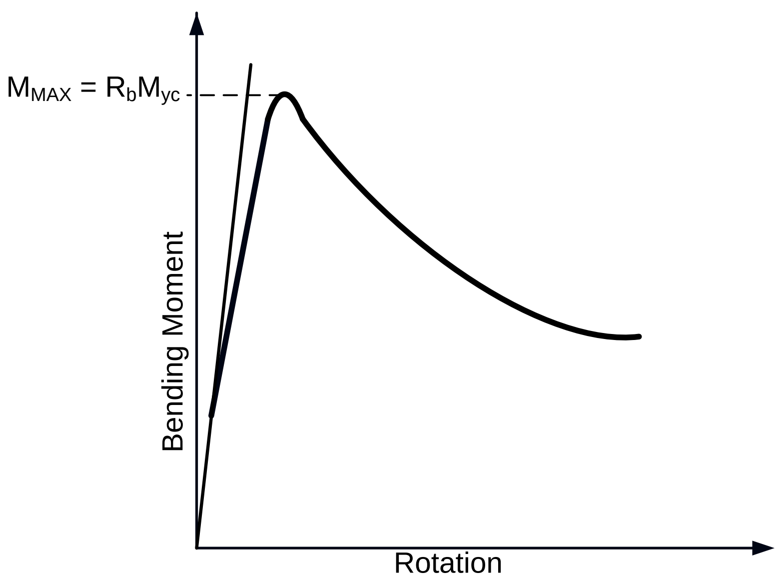

Figure 24.1-2

Moment versus Curvature for Homogeneous Slender Web Section

Sections with a web slenderness between the slenderness limits for a compact web and a slender web section are termed non-compact web sections. This represents a change from previous AASHTO Specifications, which defined sections as either compact or non-compact and did not distinguish between a non-compact and a slender web.

In *AASHTO LRFD*, a non-compact web section is defined as a non-composite section (or a composite section in negative flexure) that has a web satisfying steel grade requirements and with a slenderness at or below the limit at which theoretical elastic web bend-buckling does not occur for elastic stress levels, computed according to beam theory, smaller than the limit of the nominal flexural resistance.

Because web bend-buckling is not assumed to occur, R_b is taken equal to 1.0 for these sections. The maximum flexural resistance of a non-compact web section, M_{max}, is taken as the smaller of $R_{pc}M_{yc}$ and $R_{pt}M_{yt}$. It falls between M_{max} for a compact web and a slender web section as a linear function of the web slenderness ratio. R_{pc} and R_{pt} are termed web plastification factors for the compression and tension flange, respectively. The web plastification factors are essentially effective shape factors that define a smooth linear transition in the maximum flexural resistance between M_y and M_p.

The basic relationship between M_{max} and the web slenderness $2D_c/t_w$ given in *AASHTO LRFD* is presented in Figure 24.1-3. Figure 24.1-3 assumes that yielding with respect to the compression flange controls. The relationship between M_{max} and web slenderness is defined in terms of all three types of sections – compact web, non-compact web and slender web.

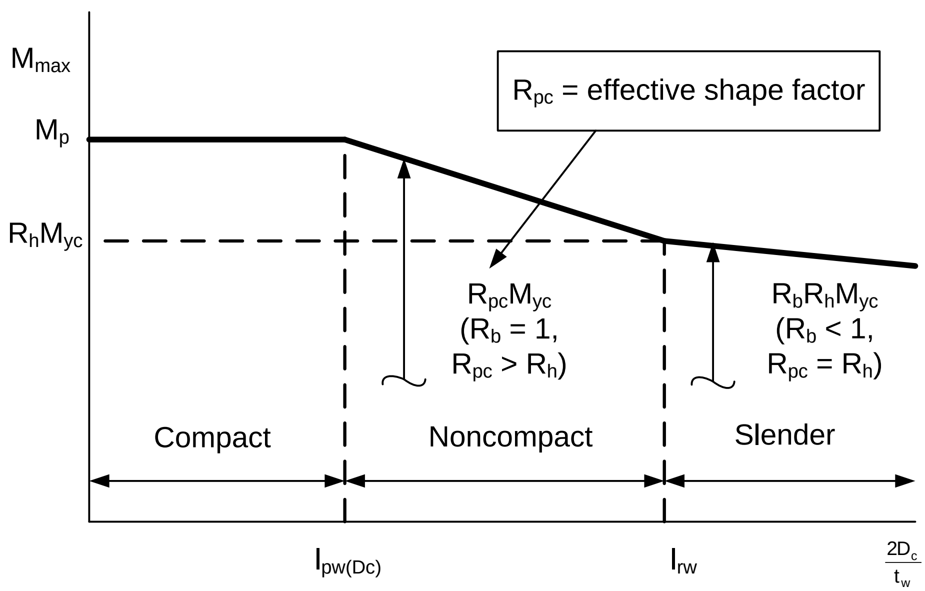

Figure 24.1-3
M_{max} versus Web Slenderness

In *AASHTO LRFD*, the flexural resistance for slender web sections is expressed in terms of stress. For compact web and non-compact web sections, in which the maximum potential flexural resistance equals or exceeds M_y, the resistance equations are more conveniently expressed in terms of bending moment.

Lateral torsional buckling can result if the compression flange of an I-section member does not have adequate lateral support. The member deflects laterally in a torsional mode before the compressive bending stress reaches the yield stress. Lateral torsional buckling is illustrated in Figure 24.1-4.

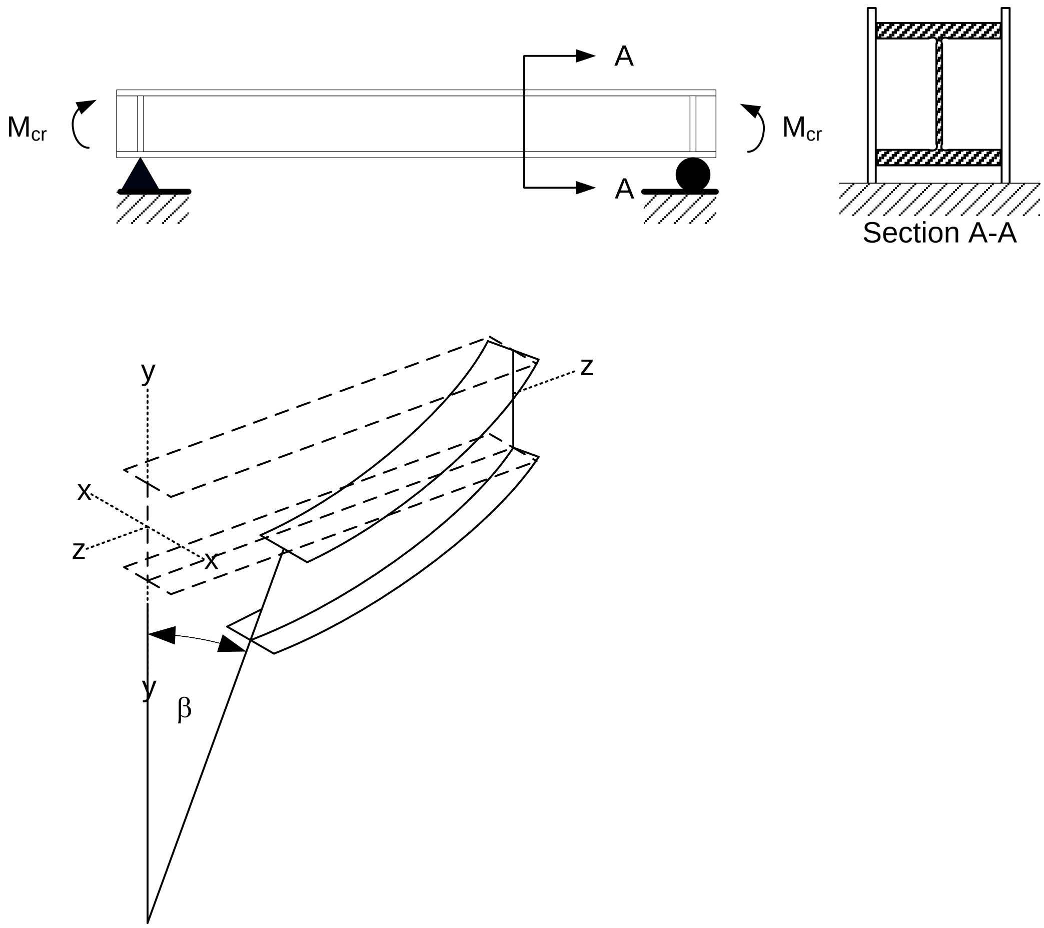

Figure 24.1-4
Lateral Torsional Buckling in a Doubly Symmetric I-section Member

As presented in Figure 24.1-5, *AASHTO LRFD* has adopted a simple linear expression to approximate the lateral-torsional buckling resistance of discretely braced compression flanges in the inelastic range. Figure 24.1-5 also shows the basic form of the flange local buckling equations in *AASHTO LRFD*, which is similar to the form of the lateral-torsional buckling equations.

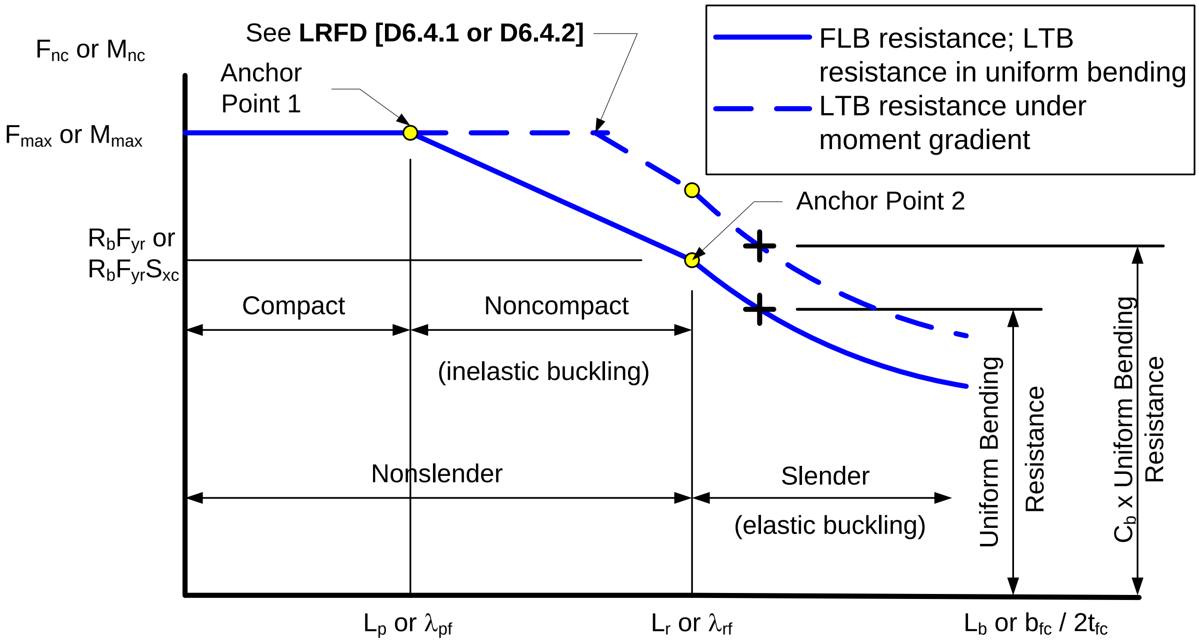

Figure 24.1-5
Form of the Compression-Flange Resistance Equations in *AASHTO LRFD*

24.2 Materials

Structural steels currently used conform to ASTM A709 Specifications designated Grades 36, 50 and 50W. *AASHTO LRFD* gives the necessary design information for each grade of steel. Steel girders may utilize High-Performance Steel (HPS); however it may come at a premium price due to the limited number of mills that are rolling HPS. The limited number of mills may also have adverse effects on the delivery schedule.

HPS is currently produced by either quenching and tempering (Q&T) or by thermo-mechanical-controlled-processing (TMCP). TMCP HPS is currently available in plate thicknesses up to 2" and in maximum plate lengths from approximately 50' to 125' depending on weights. Q&T HPS is available in plate thicknesses from 2" to 4" (or less for larger plate widths), but because of the furnaces that are used in the tempering process, it is subject to a maximum plate-length limitation of 600" (50') or less, depending on weights. Therefore, whenever Q&T HPS is used (generally when HPS plates over 2" in thickness are specified), the maximum plate-length limitation should be considered when laying out flange (and web) transitions in a girder.

For fracture toughness, HPS provides significant toughness improvements given, that by default, Charpy V-notch requirements satisfy the more stringent Zone 3 requirements in all temperature zones. For welding, most of the bridge steels specified in the ASTM A709 Specifications can be welded without special precautions or procedures. However, special procedures should be followed to improve weldability and ensure high-quality welds when HPS is used.

Hybrid girder design utilizing HPS Grade 70 steel (Grade 70 is only available in HPS) for the flanges and Grade 50 steel for the web may be considered as a viable alternative. Such an arrangement has recently proven to be a popular option, primarily in regions of negative flexure.

For unpainted structures over stream crossings, Grade 50W weathering steel is recommended throughout.

Cracks have been observed in steel girders due to fabrication, fatigue, brittle fractures and stress corrosion. To insure against structural failure, the material is tested for plane-strain fracture toughness. As a result of past experience, the Charpy V-notch test is currently required on all grades of steel used for girders.

Plate width and length availability is an important consideration when it comes to sizing girder flanges. The availability of plate material varies from mill to mill. Generally, plates are available in minimum widths ranging from 48" to 60" and in maximum widths ranging from 150" to 190". AASHTO/NSBA Steel Bridge Collaboration, "Guidelines for Design for Constructibility, G12.1" (2003) contains some example plate length and width availability information from a single mill. However, a fabricator and/or mill should be consulted regarding the most up-to-date plate availability information. The maximum available plate length is generally a function of the plate width and thickness, steel grade and production process.

For additional information about plate widths and lengths, including maximum sizes for shipping and erection, see 24.4.6.2.

For additional information about materials, see Chapter 9 – Materials.

24.2.1 Bars and Plates

Bars and plates are grouped under flat rolled steel products that are designated by size as follows:

- Bars – 8" or less in width
- Plates – over 8" in width

WisDOT policy item:

AASHTO LRFD allows a minimum thickness of 5/16” for most structural steel members. Current WisDOT policy is to employ a minimum thickness of 7/16" for primary members and a minimum of 3/8" for secondary structural steel members.

Optional splices are permitted on plates which are detailed over 60' long. Refer to the latest steel product catalogs for steel sections and rolled stock availability.

24.2.2 Rolled Sections

A wide variety of structural steel shapes are produced by steel manufacturers. Design and detail information is available in the *AISC Manual of Steel Construction*, and information on previously rolled shapes is given in *AISC Iron and Steel Beams 1873 to 1952*. Refer to the latest steel product catalogs for availability and cost, as some shapes are not readily available and their use could cause costly construction delays.

24.2.3 Threaded Fasteners

The design of bolted connections is covered in **LRFD [6.13.2]**. As specified in **LRFD [6.13.2.1]**, bolted steel parts must fit solidly together after the bolts are tightened. The bolted parts may be coated or uncoated. It must be specified in the contract documents that all joint surfaces, including surfaces adjacent to the bolt head and nut, be free of scale (except for tight mill scale), dirt or other foreign material. All material within the grip of the bolt must be steel.

High-strength bolts are installed to have a specified initial tension, which results in an initial pre-compression between the joined parts. At service load levels, the transfer of the loads between the joined parts may then occur entirely via friction, with no bearing of the bolt shank against the side of the hole. Until the friction force is overcome, the shear resistance of the bolt and the bearing resistance of the bolt hole will not affect the ability to transfer the load across the shear plane between the joined parts.

In general, high-strength bolted connections designed according to *AASHTO LRFD* will have a higher reliability than the connected parts because the resistance factors for the design of bolted connections were selected to provide a higher level of reliability than those chosen for member design. Also, the controlling strength limit state in the connected part (for example, yielding or deflection) is typically reached well before the controlling strength limit state in the connection (for example, the bolt shear resistance or the bearing resistance of the connected material).

AASHTO LRFD recognizes two types of high-strength bolted connections – slip-critical connections and bearing-type connections. The resistance of all high-strength bolted connections in transmitting shear across a shear plane between bolted steel parts is the same whether the connection is a slip-critical or bearing-type connection. The slip-critical connection has an additional requirement that slip must not occur between the joined parts at service load levels.

Slip-critical (or friction) type connections are used on bridges since the connections are subject to stress reversals and bolt slippage is undesirable. High strength bolts in friction type connections are not designed for fatigue. The allowable unit stresses, minimum spacing and edge distance as given in *AASHTO LRFD* are used in designing and detailing the required number of bolts. A490 bolts shall not be used in tension connections due to their low fatigue strength. Generally, A325 bolts are used for steel connections unless the higher strength A490 bolt is warranted. If at all possible, avoid specifying A490, Type 3 bolts on plans for unpainted structures. All bolt threads should be clean and lubricated with oil or wax prior to tightening.

Steel connections shall be made with high strength bolts conforming to ASTM designations A325 and A490. Galvanized A490 bolts cannot be substituted for A325 bolts; if A490 bolts are galvanized, failure may occur due to hydrogen embrittlement. ASTM specifications limit galvanizing to A325 or lower strength fasteners. All bolts for a given project should be from the same location and manufacturer.

High strength pin bolts may be used as an alternate to A325 bolts. The shank and head of the high strength steel pin bolt and the collar fasteners shall meet the chemical composition and mechanical property requirements of ASTM designation A325, Types 1, 2 or 3.

24.2.3.1 Bolted Connections

Bolted connections shall be designed as follows:

1. All field connections are made with 3/4" high strength bolts unless noted or shown otherwise.

2. Holes for bolted connections shall not be more than 1/16" greater than the nominal bolt diameter.

3. Faying surfaces of friction type connections are blast cleaned and free from all foreign material. Note that AASHTO LRFD allows various design stresses depending on surface condition of bolted parts.

4. Bolts are installed with a flat, smooth, hardened circular washer under the nut or bolt head, whichever element is turned in tightening the connection.

5. A smooth, hardened, bevel washer is used where bolted parts in contact exceed a 1 to 20 maximum slope.

6. Where clearance is required, washers are clipped on one side to a point not closer than seven-eighths of the bolt diameter from the center of the washer.

7. After all bolts in the connections are installed, each fastener shall be tightened equal to the proof load for the given bolt diameter as specified by ASTM A490 bolts and galvanized A325 bolts shall not be reused.

Retightening previously tightened bolts which may have been loosened by tightening of adjacent bolts is not considered a reuse.

24.2.4 Quantity Determination

For information about determining structural steel and bolt weight, see subsection 506.4 of the *State of Wisconsin Standard Specification for Highway and Structure Construction.*

For new structures, the bolt length is not required on the plans. For rehabilitation plans, when connecting new steel to existing steel, indicate either the required grip or the thickness of the existing material, in addition to the bolt diameter. Bolt weight should be included with the specified structural steel of the lower strength material being joined.

24.3 Design Specification and Data

24.3.1 Specifications

Refer to the design and construction related materials as presented in the following specifications:

1. Bridge Welding Code: AASHTO/AWS-D1.5.

2. American Institute of Steel Construction (AISC) Manual of Steel Construction.

24.3.2 Resistance

Material properties required to compute the nominal and factored resistance values are given in *AASHTO LRFD*. Information for the more common structural components used on bridges is provided in Chapter 9 - Materials.

24.3.3 References for Horizontally Curved Structures

Standard for Girder Layout on Curve shows the method for laying out kinked steel girders on horizontally curved bridges. For horizontally curved structures, girders can either be kinked at field splice locations or they can be curved throughout. Curved girders are generally preferable because they result in a constant overhang and are generally more aesthetically pleasing. For a kinked girder, lateral bending may be concentrated at the location of the kink.

For horizontally curved steel girders, **LRFD [2.5.2.6.3]** suggests that the maximum span-to-depth ratio for the steel girder be limited to ArcSpan/25 (or less depending on certain conditions). An increase in the preferred minimum depth for curved steel girders reflects the fact that the outermost curved girder receives a disproportionate share of the load and needs to be stiffer. Increasing the depth and stiffness of all the girders in a curved-bridge system leads to smaller relative deflections between girders and to smaller cross-frame forces as a result. Deeper girders also result in reduced out-of-plane girder rotations, which may make the bridge easier to erect. Similarly, in curved and straight steel bridges with skewed supports, cross-frame forces are directly related to the relative girder deflections, and increasing the girder depth and stiffness can help control the relative deflections. For additional information about cross frames and diaphragms, see 24.4.5.

24.3.4 Design Considerations for Skewed Supports

Modern highway design must recognize vehicle speed and right-of-way cost. These factors have reversed the position of the bridge designer from determining the layout of a bridge, including the approaching roadway and span arrangement, to designing bridges for a predetermined space. This allotted space may limit bridge depth, span arrangement and pier location. Additional constraints on the design include sight distances, setbacks and other constraints such as environmental and aesthetic factors. This plethora of constraining factors makes the design of bridges more challenging rather than limiting. Skewed supports are one of the most common factors introduced in modern bridge design. Spanning streams or highways not perpendicular to the bridge alignment frequently requires the introduction of skewed supports.

The engineer is best served if the skew of the supports can be reduced. Reduction of the skew often involves increasing the span, which may lead to deeper girders. When girder depth is limited, this may not be a practical solution. However, reduction of skew has the advantage of reducing abutment and/or pier length. This cost reduction should always be balanced against any increase in superstructure cost related to the use of longer spans. Simply minimizing the square footage of the bridge deck is often not the most economical solution.

One of the most problematic skew arrangements is variable skew of adjacent substructure units. This arrangement leads to different length girders with different stiffnesses, and subsequently, different vertical deflections. Hence, reduction of skew on one support while it remains on the other is not a desirable way to address skew, and such a skew arrangement should be used only as a last resort.

Multi-girder bridges are integral structures with transverse elements. Analysis of the structure must acknowledge the restoring forces in the transverse members. In multi-girder bridges with right supports and equal-stiffness girders, the action of these restoring forces is implied within the wheel-load distribution factors that are often employed. Parallel skews have equal length girders with equal stiffnesses. However, when the relative stiffness of points on adjacent girders attached by cross frames or diaphragms is different (for example, when the cross frames or diaphragms are perpendicular to the girders), the design becomes more problematic. The skew affects the analysis of these types of skewed bridges by the difference in stiffness at points connected by perpendicular cross frames.

It should be noted that dead load as well as live load is affected by skew. The specifications address the effect of skew on live load by providing correction factors to account for the effect of skew on the wheel-load distribution factors for bending moment and end support shear in the obtuse corner (see **LRFD [Table 4.6.2.2.2e-1]** and **LRFD [Table 4.6.2.2.3c-1]**, respectively). There is currently no provision requiring dead load on skewed bridges to be addressed differently than for other bridges. For additional information about the effects of skew on live load distribution factors, see 17.2.8.

The effect of skew is far from constant on all bridges. The significance of skew is increased with increasing skew with respect to the girder line, with increased deflections and in simple spans. Skewed simple spans seem to be more problematic than continuous spans with the same skew.

Arrangement of cross frames and diaphragms is challenging for sharply skewed girder bridges. If the skew is 15 degrees or less and both supports have the same skew, it is usually desirable to skew the cross frames or diaphragms to be parallel with the supports. This arrangement permits the cross frames or diaphragms to be attached to the girders at points of equal stiffness, thus reducing the relative deflection between cross frame and diaphragm ends, and thus, the restoring forces in these members. *AASHTO LRFD* permits parallel skews up to 20 degrees.

WisDOT policy item:

For skews greater than 15 degrees, the cross frames and diaphragms must be placed perpendicular to the girders.

Typically, the cross frames or diaphragms can be staggered. This arrangement reduces the transverse stiffness because the flanges flex laterally and relieve some of the force in the cross frames or diaphragms. There is a resultant increase in lateral bending moment in the flanges. Often, this lateral bending is not critical and the net result is a desirable reduction in cross-frame forces or diaphragm forces. Smaller cross-frame forces or diaphragm forces permit smaller cross-frame or diaphragm members and smaller, less expensive cross-frame or diaphragm connections. Alternatively, they are placed in a contiguous pattern with the cross frames or diaphragms matched up on both sides of the interior girders, except near the bearings. This arrangement provides the greatest transverse stiffness. Thus, cross-frame forces or diaphragm forces are relatively large, and the largest amount of load possible is transferred across the bridge. This results in the largest reduction of load in the longitudinal members (that is, the girders). The bearings at oblique points receive increased load.

The exterior girders always have cross frames or diaphragms on one side, but since there are no opposing cross frames or diaphragms on the other side, lateral flange bending is usually small in these girders, which often have critical vertical bending moments compared to the interior girders. Interior girders are generally subjected to larger lateral flange bending moments when a staggered cross-frame arrangement is employed.

In lieu of a refined analysis, **LRFD [C6.10.1]** contains a suggested estimate of 10.0 ksi for the total unfactored lateral flange bending stress, f_λ, due to the use of discontinuous cross-frame or diaphragm lines in conjunction with a skew angle exceeding 15 degrees. It is further suggested that this value be proportioned to dead and live load in the same proportion as the unfactored major-axis dead and live load bending stresses. It is currently presumed that the same value of the flange lateral buckling, f_λ, should be applied to interior and exterior girders, although the suggested value is likely to be conservative for exterior girders for the reason discussed previously. Therefore, lateral flange bending due to discontinuous cross-frame lines in conjunction with skew angles exceeding 15 degrees is best handled by a direct structural analysis of the bridge superstructure.

At piers, it is usually not necessary to use a cross-frame or diaphragm line along the pier. Nor is it necessary to have a cross frame or diaphragm at each bearing. It is necessary to have a perpendicular cross frame or diaphragm at each bearing that is fixed laterally in order to transfer loads into the bearing. Otherwise, lateral bending in the bottom flange is excessive. Some means should be provided to allow for jacking the girder to replace bearings. At abutments and other simple supports, a row of cross frames or diaphragms is always required to support the free edge of the deck. The end rotation of the girders creates forces in these cross frames or diaphragms, which in turn create end moments in the girders. Usually the end moments are negative. Note that the larger the rotation and deflection of the girders, the larger the end moments. In some cases, these end moments are important. Generally, they cannot be avoided. However, by placing the deck at the ends of the bridge last, the tensile stresses in the deck can be minimized.

Differential deflections between the ends of the cross frames in skewed bridges along with differential rotations of the girders (about an axis transverse to the longitudinal axis of the girders) result in twist of the girders, which can make girder erection and fit-up of the cross-frame connections more difficult as the dead load is applied. As discussed in **LRFD [C6.7.2]**, in order for the girder webs of straight skewed I-girder bridges to end up theoretically vertical (or plumb) at the bearings under either the steel or full dead load condition, the cross frames or diaphragms must be detailed for that condition in order to introduce the necessary reverse twist into the girders during the erection so that the girders will rotate back to a theoretically plumb position as the corresponding dead load is applied. The steel dead load condition refers to the condition after the erection of the steel is completed. The full dead load condition refers to the condition after the full non-composite dead load, including the concrete deck, is applied. The cross frames or diaphragms may have to be forced into position in this case, but this can usually be accomplished in straight skewed I-girder bridges without inducing significant locked-in stresses in the girder flanges or the cross frames or diaphragms. The twist, ϕ, of the girders at the end supports in a straight skewed I-girder bridge can either be determined from a refined analysis, or it can be approximated from the following equation:

$$\phi = \frac{\left[\mathrm{Sin}\left(\mathrm{Tan}^{-1}\theta\right)d\right]}{\mathrm{Tan}\ \alpha}$$

Where:

α = Skew angle of the end support measured with respect to the longitudinal axis of the girder (radians)

θ = Girder end rotation due to the appropriate dead load about an axis transverse to the longitudinal axis of the girder (radians)

d = Girder depth (in.)

Alternatively, the girders may be erected in the no-load condition (that is, the condition where the girders are erected plumb under a theoretically zero-stress condition neglecting any stress due to the weight of the steel acting between points of temporary support), with the cross frames or diaphragms detailed to fit theoretically stress-free. In this case, the girders will rotate out-of-plumb as the corresponding dead load is applied. Therefore, the engineer should consider the effect of any potential errors in the horizontal roadway alignment under the full dead load condition resulting from the girder rotations. Also, it should be ensured that the rotation capacity of the bearings is sufficient to accommodate the twist or that the bearings are installed so that their rotation capacities are not exceeded.

For straight skewed I-girder bridges, **LRFD [6.7.2]** requires that the contract documents clearly state an intended erected position of the girders (that is, either girder webs theoretically plumb or girder webs out-of-plumb) and the condition under which that position is to be theoretically achieved (that is, either the no-load condition, steel dead load condition or full dead load condition). The provisions of **LRFD [2.5.2.6.1]** related to bearing rotations for straight skewed I-girder bridges are also to be applied. These provisions are intended to ensure that the computed girder rotations at bearings for the accumulated factored loads corresponding to the engineer's assumed construction sequence do not exceed the specified rotational capacity of the bearings.

It should be apparent that all of the issues relating to skewed bridges are related to deflection. The smaller the deflections, both dead load and live load, the less critical are the above issues. Thus, deep girders and low design stresses are beneficial to skewed bridges.

For additional information about bracing, including cross frames and diaphragms, see 24.4.5.

24.4 Design Considerations

Steel girder structures are analyzed and designed using LRFD. *AASHTO LRFD* provides the details for designing simple and continuous steel girders for various span lengths using LRFD.

WisDOT Policy Item:

Do no utilize optional **LRFD (Appendix A6)** providing Flexural Resistance of Straight Composite I-Sections in Negative Flexure and Straight Non-composite I-Sections with Compact or Non-compact Webs.

Design considerations common to all superstructure types, including distribution of loads, dead load, traffic live load, pedestrian load and wind load, are presented in Chapter 17 – Superstructures - General.

24.4.1 Design Loads

24.4.1.1 Dead Load

For steel girder structures, dead loads should be computed based on the following:

1. The weight of the concrete haunch is determined by estimating the haunch depth at 2-1/2" and the width equal to a weighted average of the top flange width.

2. The weight of steel beams and girders is determined from the AISC Manual of Steel Construction. Haunched webs of plate girders are converted to an equivalent uniform partial dead load.

3. The weight of secondary steel members such as bracing, shear studs and stiffeners can be estimated at 30 plf for interior girders and 20 plf for exterior girders.

4. A dead load of 20 psf carried by the composite section is added to account for a future wearing surface.

AASHTO LRFD specifies that the effect of creep is to be considered in the design of composite girders which have dead loads acting on the composite sections. As specified in **LRFD [6.10.1.1.1a]** and **LRFD [6.10.1.1.1b]**, for the calculation of the stresses in a composite girder, the properties of the steel section alone should be used for permanent loads applied before the concrete deck has hardened or is made composite. The properties of the long-term 3n composite section should be used for permanent loads applied after the concrete deck has hardened or is made composite. The properties of the short-term n composite section should be used for transient loads applied after the concrete deck is made composite. **LRFD [6.10.1.1.1d]** requires that n be used to compute concrete deck stresses due to all permanent and transient loads.

Information regarding dead load deflections is given in 24.4.8

24.4.1.2 Traffic Live Load

For information about LRFD traffic live load, see 17.2.4.2.

24.4.1.3 Pedestrian Live Load

For information about LRFD pedestrian live load, see 17.2.4.4.

24.4.1.4 Temperature

Steel girder bridges are designed for a coefficient of linear expansion equal to .0000065/°F at a temperature range from -30 to 120°F. Refer to Chapter 28 – Expansion Devices for expansion joint requirements, and refer to Chapter 27 – Bearings for the effect of temperature forces on bearings.

24.4.1.5 Wind

For information about LRFD wind load, see Chapter 17 – Superstructures - General. In addition, see 24.6.16 for wind effects on girder flanges and 24.6.22 for design of bracing.

24.4.2 Minimum Depth-to-Span Ratio

Traditional minimum depths for constant depth superstructures are provided in **LRFD [Table 2.5.2.6.3-1]**. For steel simple-span superstructures, the minimum overall depth of the composite girder (concrete slab plus steel girder) is 0.040L and the minimum depth of the I-beam portion of the composite girder is 0.033L. For steel continuous-span superstructures, the minimum overall depth of the composite girder (concrete slab plus steel girder) is 0.032L and the minimum depth of the I-beam portion of the composite girder is 0.027L. For trusses, the minimum depth is 0.100L.

For a given span length, a preliminary, approximate steel girder web depth can be determined by referring to Table 24.4-1. This table is based on previous design methods and should therefore be used for preliminary purposes only. However, it remains a useful tool for approximating an estimated range of web depths for a given span length. Recommended web depths are given for parallel flanged steel girders. The girder spacings and web depths were determined from an economic study, deflection criteria and load-carrying capacity of girders for a previous design method.

From a known girder spacing, the effective span is computed as shown in Figure 17.5-1. From the effective span, the slab depth and required slab reinforcement are determined from tables in Chapter 17 – Superstructures - General, as well as the additional slab reinforcement required due to slab overhang.

10' Girder Spacing, 9" Deck		12' Girder Spacing, 10" Deck	
Span Lengths (Ft.)	Web Depth (In.)	Span Lengths (Ft.)	Web Depth (In.)
90 – 115	48	90 – 103	48
116 – 131	54	104 – 119	54
132 – 140	60	120 – 127	60
141 – 149	66	128 – 135	66
150 – 163	72	136 – 146	72
164 – 171	78	147 – 153	78
172 – 180	84	154 – 163	84
181 – 190	90	164 – 170	90
191 – 199	96	171 – 177	96
200 – 207	102	178 – 184	102
208 – 215	108	185 – 192	108

Table 24.4-1
Parallel Flange Girder Recommended Depths
For 2-Span Bridges with Equal Span Lengths)

24.4.3 Live Load Deflections

WisDOT requirements for allowable live load deflection are described in 17.2.12, and the computation of actual live load deflection is explained in 17.2.13.

Limiting the live load deflection ensures a minimum degree of stiffness in the steel girders and helps when constructing the bridge. This is especially important when using higher-strength high-performance steels which can result in shallower and more flexible girders, particularly on curved and/or skewed bridges.

24.4.4 Uplift and Pouring Diagram

Permanent hold-down devices are used to attach the superstructure to the substructure at the bearing when any combination of loading using Strength I loading combination (see **LRFD [C3.4.1]**) produces uplift. Also, permanent hold-down devices are required on alternate girders that cross over streams with less than 2' clearance for a 100-year flood where expansion bearings are used. These devices are required to prevent the girder from moving off the bearings during extreme flood conditions.

Uplift generally occurs under live loading on continuous spans when the span ratio is greater than 1 to 1.75. However, a span ratio of 1.75 should be avoided. Under extreme span ratios, the structure may be in uplift for dead load. When this occurs, it is necessary to jack the girders upward at the bearings and insert shim plates to produce a downward dead load reaction. The use of simple spans or hinged continuous spans is also considered for this case.

On two-span bridges of unequal span lengths, the slab is poured in the longer span first. Cracking of the concrete slab in the positive moment region has occurred on bridges with extreme span ratios when the opposite pouring sequence has been followed. When the span exceeds 120', consider some method to control positive cracking such as limited pouring time, the use of retarders and sequence of placing.

On multiple-span structures, determine a pouring sequence that causes the least structure deflections and permits a reasonable construction sequence. Refer to Standard for Slab Pouring Sequence for concrete slab pouring requirements. Temporary hold-down devices are placed at the ends of continuous girders where the slab pour ends if permanent hold-down devices are not required. The temporary hold-down devices prevent uplift and unseating of the girders at the bearings during the pouring sequence. Consideration should be given to including temporary hold-down devices at the end of the bridge where deck removal begins on deck replacement projects.

Standard hold-down devices having a capacity of 20 kips are attached symmetrically to alternate girders or to all the girders as required. Hold-down devices are designed by considering line bearing acting on a pin. Refer to Standard for Hold Down Devices for permanent and temporary hold-down details. To compute uplift, a shear influence line is first obtained. Next the wheel load distribution factor is determined in the same manner as for live load deflection. The number of loaded lanes is based on the width of the bridge between curbs. The live load plus dynamic load allowance is uniformly distributed to all the girders and is adjusted based on the appropriate multiple presence factor (see **LRFD [3.6.1.1.2]**). The live load is increased 100 percent and applied to the shear influence line to produce maximum uplift. The allowance for future wearing surface should not be included in uplift computations when this additional dead load increases the end reaction.

For additional information about construction and constructability verifications, see 24.12.

24.4.5 Bracing

All bracing systems must be attached to the main girder connection stiffener by bolted connections.

24.4.5.1 Intermediate Diaphragms and Cross Frames

Diaphragms or cross frames are required at each support and at regular intervals throughout the span in all bays. Although not explicitly stated in *AASHTO LRFD*, a common rule of thumb, based on previous editions of the *AASHTO Specifications*, is to use a maximum cross-frame spacing of 25 feet. The cross-frame spacing can affect the required flange thicknesses, as well as constructability checks for stability before the deck is cured. Currently, stay-in-place forms should not be considered to provide adequate bracing to the top flange.

The spacing should be adjusted to miss any splice material. The transverse bracing is placed parallel to the skew for angles up to and including 15 degrees. Transverse bracing is placed normal to the girders for skew angles greater than 15 degrees. When diaphragms are stepped slightly out of straight through alignment, the girder flanges will experience the greatest torsional stress. Larger steps in diaphragm spacing allow the torsional moment to distribute over a longer girder section. On curved girder structures, the diaphragms are placed straight through radial lines to minimize the effects of torsion since the diaphragms or cross frames are analyzed as primary load-carrying members.

Diaphragm details and dimensions are given on Standards for Plate Girder Diaphragms & Cross Frames and Rolled Girder Diaphragms. Diaphragms carry moment and tensile stresses caused by girder deflections. In the composite slab region, the steel section acts similar to the lower chord of a vierendeel truss and is in tension. A rigidly connected diaphragm resists bending due to girder deflection and tends to distribute the load. It is preferable to place diaphragms at the 0.4 point of the end spans on continuous spans and at the center of interior spans when this can be accomplished without an increase in total number. Also, if practical, place diaphragms adjacent to a field splice between the splice and the pier. Bolted diaphragm connections are used in place of welded diaphragm connections. All cross framing is attached to this main girder connection stiffener using bolted gusset plates.

Cross framing is used for web depths over 48". The bracing consists of two diagonal members connected at their intersection and one bottom chord member. The bottom chord is designed as a secondary compression member. The diagonals are designed as secondary tension members. The length of a minimum 1/4" fillet weld size is determined for each member based on a minimum of 75 percent of the member strength.

On spans over 200' in length, the stresses caused by wind load on part of the erected girders without the slab in place may control the size of the members. Construction loads are also considered in determining member size.

On girders where longitudinal stiffeners are used, the relative position of the stiffener to the cross frame is checked. When the longitudinal stiffener interferes with the cross frame, cope the gusset plate attached to the vertical stiffener and attach the cross frame to the gusset plates, as shown in Figure 24.4-1.

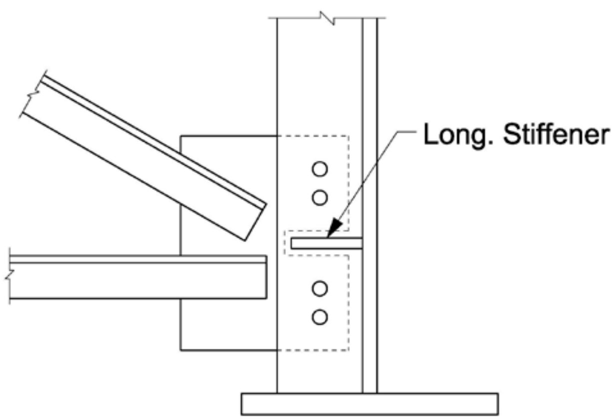

Figure 24.4-1
Cross Frame Where Longitudinal Stiffener is Used

24.4.5.2 End Diaphragms

End diaphragms are placed horizontally along the abutment end of beams or girders and at other points of discontinuity in the structure. Channel sections are generally used for end diaphragms, and they are designed as simply-supported edge beams. The live load moment plus dynamic load allowance is determined by placing one wheel load or two wheel loads 4' apart and correcting for the skew angle at the center line of the member. Generally, the dead load moment of the overlying slab and diaphragm is insignificant and as such is neglected. End diaphragm details and dimensions are given on Standard for End Diaphragms.

End diaphragms are either bolted or welded to gussets attached to the girders at points of discontinuity in the superstructure. The gusset plates are bolted to the bearing stiffeners. The same connection detail is used throughout the structure. The connections are designed for shear only where joined at a web since very little moment is transferred without a flange connection. The connection is designed for the shear due to live load plus dynamic load allowance from the wheel loads.

24.4.5.3 Lower Lateral Bracing

Lateral bracing requirements for the bottom flanges are to be investigated. Bureau of Structures (BOS) practice is to eliminate the need for bracing by either increasing flange sizes or reducing the distance between cross frames. The controlling case for this stress is usually at a beam cutoff point. At cutoff points, the condition of maximum stress exists with the smallest flange size, where wind loads have the greatest effect. A case worth examining is the temporary stress that exists in top flanges during construction. Top flange plates, which are often only 12" wide, can be heavily stressed by wind load. A temporary bracing system placed by the contractor may be in order.

On an adjacent span to one requiring lower lateral bracing, the bracing is extended one or two panel lengths into that span. The lower lateral bracing system is placed in the exterior bays of the bridge and in at least 1/3 of the bays of the bridge. On longer spans, the stresses caused by wind load during construction will generally govern the member size.

Curved girders in Wisconsin generally do not have extremely long span lengths, and the curvature of the girders forms an arch which is usually capable of resisting the wind forces prior to placing the slab.

24.4.6 Girder Selection

The exterior girder section is always designed and detailed such that it is equal to or larger than the interior girder sections. Guidelines for ratios of girder depth to length of span are provided in 24.4.2. The following criteria are used to determine the selection and sizes of girder sections. For additional rules of thumb regarding economical design considerations, see 24.6.2.

24.4.6.1 Rolled Girders

Rolled girders without cover plates are preferred. Cover plates are not recommended due to fatigue considerations and higher fabrication costs.

24.4.6.2 Plate Girders

Basic cross-section proportion limits for flanges of steel I-girders are specified in **LRFD [6.10.2.2]**. The limits apply to both tension and compression flanges. The minimum width of flanges, b_f, is specified as:

$$b_f \geq D/6$$

Where:

D = Web depth

This limit is a lower limit, and flange widths should not be set based on this limit. Practical size flanges should easily satisfy this limitation based on satisfaction of other design criteria. Fabricators prefer that flange widths never be less than 12" to prevent distortion and cupping of the flanges during welding, which sets a practical lower limit.

Composite design has led to a significant reduction in the size of compression flanges in regions of positive flexure as economical composite girders normally have smaller top flanges than bottom flanges. In regions of positive flexure during deck placement, more than half the web is typically in compression. As a result, maximum moments generated during the deck-casting sequence, coupled with top compression flanges that are too narrow, can lead to out-of-plane distortions of the compression flanges and web during construction. The following relationship from **LRFD [C6.10.3.4]** is a suggested guideline on the minimum top compression flange width, b_{fc}, that should be provided in these regions to help minimize potential problems in these cases:

$$b_{fc} \geq L/85$$

Where:

L = Length of the girder shipping piece

Satisfaction of this simple guideline can also help ensure that individual field sections will be stable for handling both in the fabrication shop and in the field. Adherence to this guideline can also facilitate erection without any required special stiffening trusses or falsework. It is recommended that the above two equations be used to establish a minimum required top-flange width in regions of positive flexure in composite girders.

As a practical matter, fabricators order flange material from wide plate, typically between 72" and 96" wide. They either weld the shop splices in the individual flanges after cutting them to width or they weld the different thickness plates together to form one wide plate and then strip the individual flanges. In the latter case, the individual flange widths must be kept constant within an individual shipping piece, which is preferred. Changing of flange widths at shop splices should be avoided if at all possible. Stripping the individual flanges from a single wide plate allows for fewer weld starts and stops and results in only one set of run-on and run-off tabs. It is estimated that up to 35% of the labor required to join the flanges can be saved by specifying changes in thickness rather than width within a field section.

A fabricator will generally order plate with additional width and length for cutting tolerance, sweep tolerance and waste. Waste is a particular concern when horizontally curved flanges are cut curved. The engineer should give some consideration as to how the material might be ordered and spliced; a fabricator can always be consulted for assistance. Flanges should be sized (including width, thickness and length) so that plates can be ordered and spliced with minimal waste. *AASHTO/NSBA Steel Bridge Collaboration*, "Guidelines for Design for Constructability, G12.1" (2003) is a free publication available from AASHTO which contains some specific recommendations and illustrative examples related to this issue.

The following additional guidelines are used for plate girder design and detailing:

1. Maximum change in flange plate thickness is 1" and preferably less.

2. The thinner plate is not less than 1/2 the thickness of the thicker flange plate.

3. Plate thicknesses are given in the following increments:

4. 1/16" up to 1"

5. 1/8" between 1" and 2"

6. 1/4" above 2"

7. Minimum plate size on the top flange of a composite section in the positive moment region is variable depending on the depth of web, but not less than 12" x ¾" for web depths less than or equal to 66" and 14" x ¾" for web depths greater than 66". Thinner plates become wavy and require extra labor costs to straighten within tolerances.

8. For plate girder flange widths, use 2" increments.

9. For plate girder web depths, use 3" increments.

10. Changes in plate widths or depths are to follow recommended standard transition distances and/or radii. The minimum size flange plates of 16" x 1 1/2" at the point of maximum negative moment and 16" x 1" for the bottom flange at the point of maximum positive moment are recommended for use on plate girders. The use of a minimum flange width on plate girders is necessary to maintain adequate stiffness in the girder so it can be fabricated, transported and erected. Deeper web plates with small flanges may use less steel, but they create problems during fabrication and construction. However, flange sizes on plate girders with web depths 48" or less may be smaller.

11. Flange plate sizes are detailed based on recommended maximum span lengths given in Table 24.4-1 for parallel flanged girders. The most economical girder is generally the one having the least total weight but is determined by comparing material costs and welding costs for added stiffener details. Plates over 60'-90’ (depending on thickness and material) are difficult to obtain, and butt splices are detailed to limit flange plates to these lengths or less. It is better to detail more flange butt splices than required and leave the decision to utilize them up to the fabricator. All butt splices are made optional to the extent of available lengths, and payment is based on the plate sizes shown on the plans. As previously described, detail flange plates to the same width and vary the thicknesses. This allows easier fabrication when cutting plate widths. Change widths, if necessary, only at field splices.

12. Minimum web thickness is 7/16" for girder depths less than or equal to 60". An economical web thickness usually has a few transverse stiffeners. Refer to 24.10 for transverse stiffener requirements. Due to fatigue problems, use of longitudinal stiffeners for plate girders is not encouraged.

24.4.7 Welding

Welding design details shall conform to current requirements of *Bridge Welding Code: AASHTO/AWS-D1.5*. Weld details are not shown on the plans but are specified by using standard symbols as given on Figure 24.4-2 and Figure 24.4-3. Weld sizes are based on the size required due to stress or the minimum size for plate thicknesses being connected.

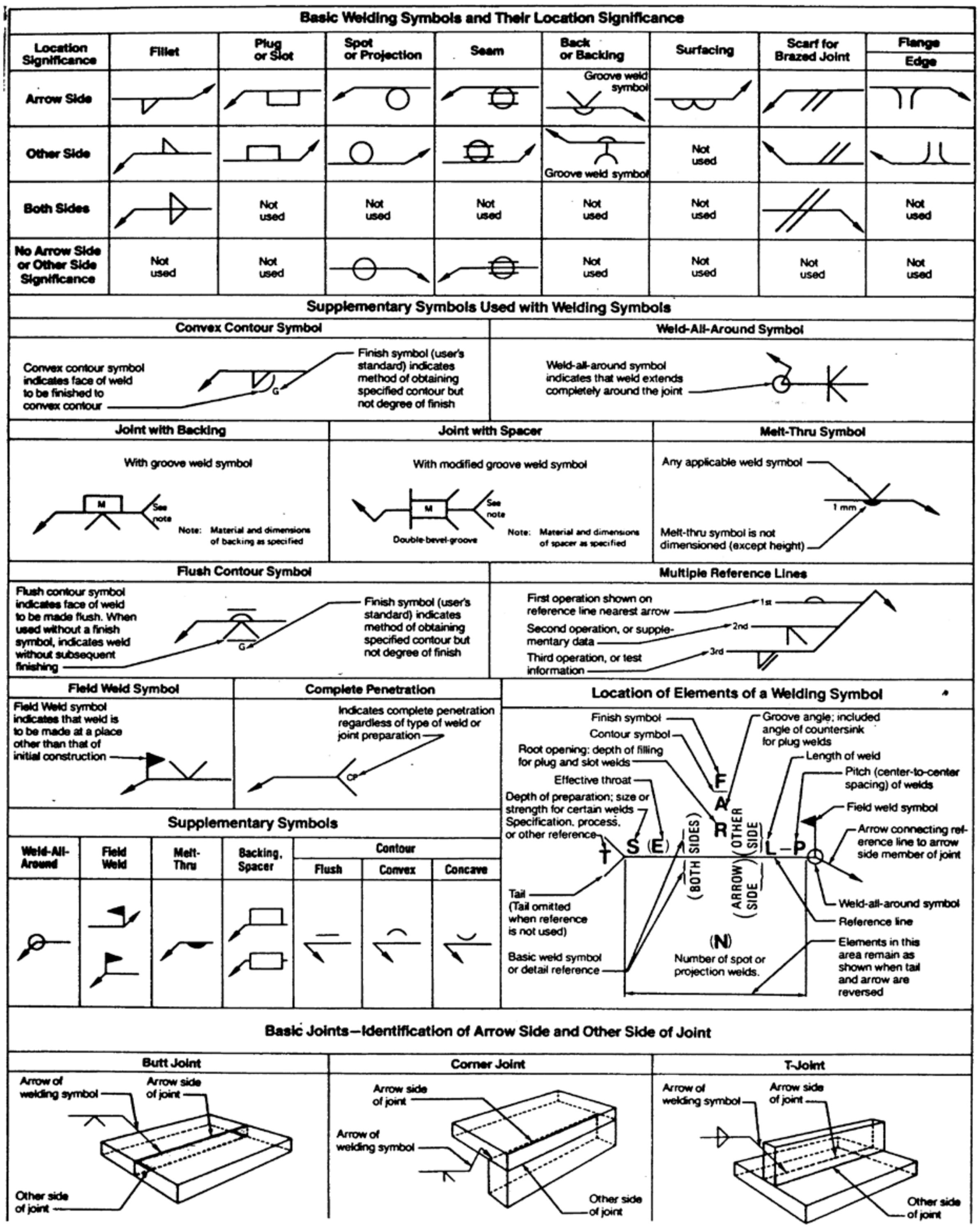

Figure 24.4-2
Basic Welding Symbols

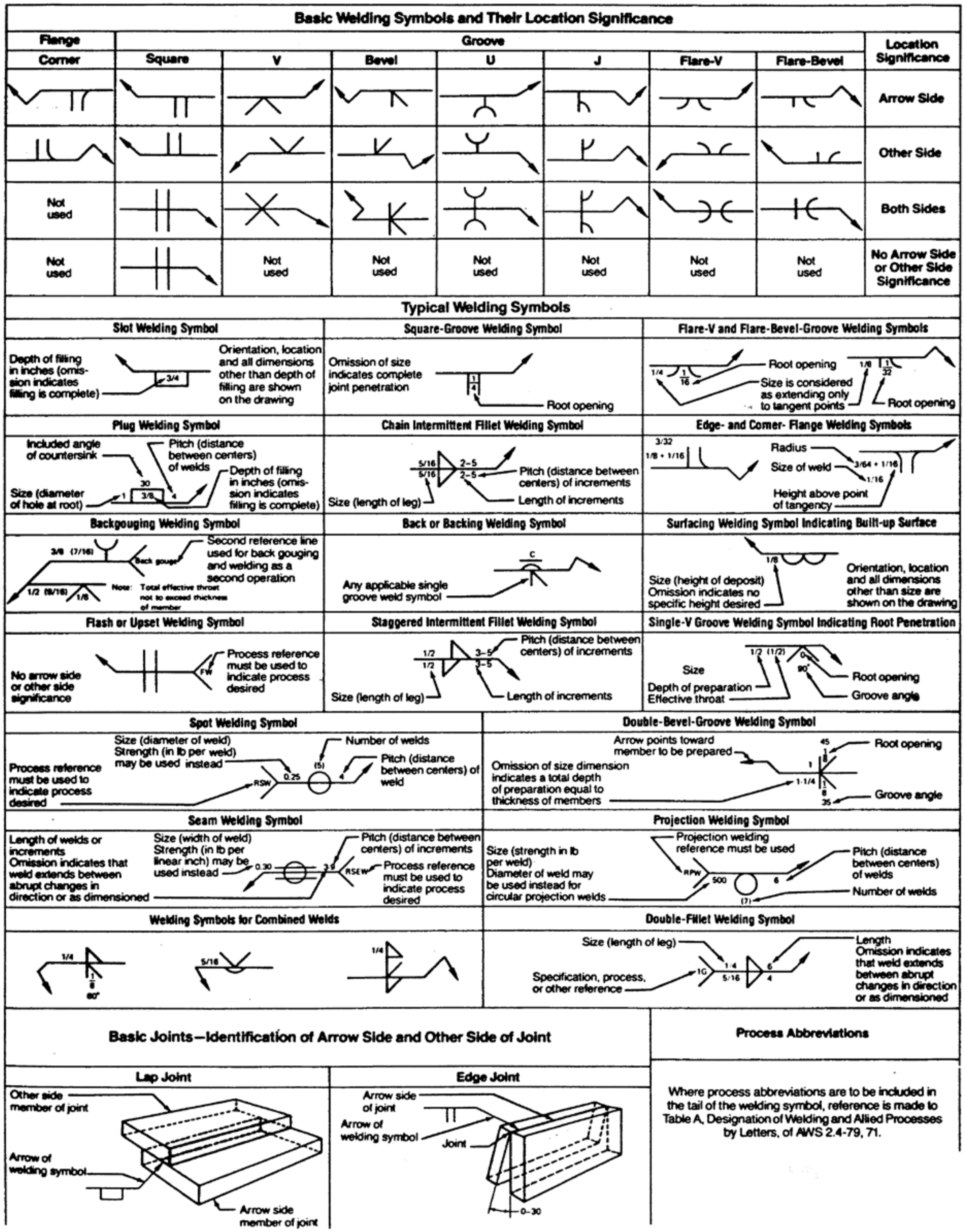

Figure 24.4-3
Basic Welding Symbols (Continued)

Fillet welds are the most widely used welds due to their ease of fabrication and overall economy. Fillet welds generally require less precision during fit-up, and the edges of the joined pieces seldom need special preparation such as beveling or squaring. Fillet welds have a triangular cross section and do not fully fuse the cross-sectional area of the parts they join, although full-strength connections can be developed with fillet welds.

The size of a fillet weld is given as the leg size of the fillet. The effective area of a fillet weld is taken equal to the effective length of the weld times the effective throat (**LRFD [6.13.3.3]**). The effective length is to be taken as the overall length of the full-size fillet. The effective throat dimension of a fillet weld is nominally the shortest distance from the joint root to the weld face, which for a typical fillet weld with equal legs of nominal size, a, is taken equal to 0.707a.

When placing a fillet weld, the welder builds up the weld to the full dimension as near to the beginning of the weld as possible. However, there is always a slight tapering off of the weld where the weld starts and ends. Therefore, a minimum effective length of the weld is required. As specified in **LRFD [6.13.3.5]**, the minimum effective length of a fillet weld is to be taken as four times its leg size, but not less than 1.5 inches.

As specified in **LRFD [6.13.3.4]**, maximum thickness (size) requirements for fillet welds along edges of connected parts depend on the thickness of the parts being connected (unless the weld is specifically designated on the contract documents to be built out to obtain full throat thickness).

The minimum thickness (size) of a fillet weld is based on the thickness of the thicker part joined, as specified on Standard for Plate Girder Details and in Table 24.4-2.

Base Metal Thickness of Thicker Part Joined	Minimum Size of Fillet Weld
Up to ½”	3/16”
Over ½” to ¾”	1/4”
Over ¾” to 1½”	5/16”
Over 1½” to 2¼”	3/8”
Over 2¼” to 6”	1/2”

Table 24.4-2
Minimum Size of Fillet Welds

The fillet weld size is not required to exceed the thickness of the thinner part joined. Refer to *AASHTO LRFD* for minimum effective fillet weld length and end return requirements.

According to **LRFD [6.13.3.2.4a]**, the factored resistance, R_r, of fillet-welded connections at the strength limit state subject to tension or compression parallel to the axis of the weld is to be taken as the corresponding factored resistance of the base metal. Note that fillet welds joining component elements of built-up members (such as girder flange-to-web welds) need not be designed for the tensile or compressive stress in those elements parallel to the axis of the welds. According to **LRFD [6.13.3.2.4b]**, the factored resistance, R_r, of fillet-welded connections at the strength limit state subject to shear on the effective area is to be taken as follows:

$$Rr = 0.6\phi_{e2}F_{exx}$$

Where:

ϕ_{e2}	=	Resistance factor for shear on the throat of the weld metal in fillet welds specified in **LRFD [6.5.4.2]** (= 0.80)
F_{exx}	=	Classification strength of the weld metal (ksi) (for example, for E70 weld metal, F_{exx} = 70 ksi)

If a certain size fillet weld must be used in adjacent areas of a particular joint, it is desirable to use the same size weld to allow the same electrodes and welding equipment to be used for that joint and to simplify the inspection.

24.4.8 Dead Load Deflections, Camber and Blocking

Show the total dead load deflections, including composite dead load (without future wearing surface) acting on the composite section, at tenth points of each span. Distribute the composite dead load evenly to all girders and provide one deflection value for a typical interior girder. Chapter 17 – Superstructure-General illustrates three load cases for exterior girder design with raised sidewalks, cases that provide a conservative envelope to ensure adequate girder capacity. However, the above composite dead load distribution should be used for deflection purposes. Total deflections and deflections for concrete only are computed to the nearest 0.1" and shown on a deflection diagram.

A separate deflection value for interior and exterior girders *may* be provided if the difference, accounting for load transfer between girders, warrants multiple values. A weighted distribution of composite dead load could be used for deflection purposes *only*. For example, an extremely large composite load over the exterior girder could be distributed as 40-30-30 percent to the exterior and first two interior girders respectively. Use good engineering judgment whether to provide separate deflection values for individual girder lines. In general, this is not necessary.

When straight girder sections between splice joints are erected, final girder elevations usually vary in height between the girder and roadway elevations due to dead load deflections and vertical curves. Since a constant slab thickness is detailed, a concrete haunch between the girder and slab is used to adjust these variations. If these variations exceed 3/4", the girder is cambered to reduce the variation of thickness in the haunch. This is done for all new girders, including widenings. Straight line chords between splice points are sometimes used to create satisfactory camber. If separate deflections are required for exterior girders, as described in Chapter 6 – Plan Preparation and Chapter 17 – Superstructure-General, provide only one camber value for all girders that is a best fit.

Welded girders are cambered by cutting the web plates to a desired curvature. During fabrication, all web plates are cut to size since rolled plates received from the mill are not straight. There is a problem in fabricating girders that have specified cambers less than 3/4", so they are not detailed.

Rolled sections are cambered by the application of heat in order that less camber than recommended by AISC specifications may be used. The concrete haunch is used to control the remaining thickness variations.

A blocking diagram is given for all continuous steel girder bridges on a vertical curve. Refer to Standard for Blocking & Slab Haunch Details for blocking and slab haunch details. Blocking heights to the nearest 1/16" are given at all bearings, field splices and shop splice points. The blocking dimensions are from a horizontal base line passing through the lower end of the girder at the centerline of bearing.

The plans should show in a table the top of steel elevations after erection at each field splice and at the centerline of all bearings.

It should be noted that the plans are detailed for horizontal distances. The fabricator must detail all plates to the erected position considering dead loads. Structure erection considerations are three-dimensional, considering slope lengths and member rotation for member end cuts.

24.4.9 Expansion Hinges

The expansion hinge as shown on Standard for Expansion Hinge Joint Details is used where pin and hanger details were previously used. The expansion hinge is more redundant and, if necessary, the bearings can easily be replaced.

24.5 Repetitive Loading and Toughness Considerations

AASHTO LRFD specifies requirements for repetitive loading and toughness considerations. Fatigue design and detail guidelines are provided, and material impact testing for fracture toughness is required. These requirements are based on performance evaluations over the past several decades on existing highways and bridges under the effects of repetitive vehicle loading.

The direct application of fatigue specifications to main load-carrying members has generally been apparent to most bridge designers. Therefore, main members have been designed with the appropriate details. However, fatigue considerations in the design of secondary members and connections have not always been so obvious. Many of these members interact with main members and receive more numerous cycles of load at a higher level of stress range than assumed. As a result, most of the fatigue problems surfacing in recent years have involved cracking initiated by secondary members.

24.5.1 Fatigue Strength

In *AASHTO LRFD*, fatigue is defined as the initiation and/or propagation of cracks due to repeated variation of normal stress with a tensile component. The fatigue life of a detail is defined as the number of repeated stress cycles that results in fatigue failure of a detail, and the fatigue design life is defined as the number of years that a detail is expected to resist the assumed traffic loads without fatigue cracking. In *AASHTO LRFD*, the fatigue design life is based on either Fatigue I for infinite load-induced fatigue life or Fatigue II for finite load-induced fatigue life.

WisDOT Policy Item

Only consider the Fatigue I limit state for steel design.

The main factors governing fatigue strength are the applied stress, the number of loading cycles and the type of detail. The designer has the option of either limiting the stress range to acceptable levels or choosing details which limit the severity of the stress concentrations.

Details involving connections that experience fatigue crack growth from weld toes and weld ends where there is high stress concentration provide the lowest allowable stress range. This applies to both fillet and groove welded details. Details which serve the intended function and provide the highest fatigue strength are recommended.

Generally, details involving failure from internal discontinuities such as porosity, slag inclusion, cold laps and other comparable conditions will have a high allowable stress range. This is primarily due to the fact that geometrical stress concentrations at such discontinuities do not exist, other than the effect of the discontinuity itself.

AASHTO LRFD provides the designer with eight basic design range categories for redundant and non-redundant load path structures. The stress range category is selected based on the highway type and the detail employed. The designer may wish to make reference to *Bridge Fatigue Guide Design and Details*, by John W. Fisher.

24.5.2 Charpy V-Notch Impact Requirements

Recognizing the need to prevent brittle fracture failures of main load-carrying structural components, AASHTO adopted provisions for Charpy V-Notch impact testing in 1974. Impact testing offers an important measure of material quality, particularly in terms of ductility. Brittleness is detected prior to placing the material in service to prevent member service failures. Wisconsin *Standard Specifications for Highway and Structure Construction* require Charpy V-Notch tests on all girder flange and web plates, flange splice plates, hanger bars, links, rolled beams and flange cover plates. Special provisions require higher Charpy V-Notch values for non-redundant structure types.

For the Charpy V-Notch impact test, small, notched steel specimens are loaded at very high strain rates as the specimen absorbs the impact from a pendulum. The maximum height the pendulum rises after impact measures the amount of energy absorbed in foot-pounds.

The AASHTO fracture control plan uses three different temperature zones (designated Zones 1, 2 and 3) to qualify the fracture toughness of bridge steels. The three zones are differentiated by their minimum operating (or service) temperatures, which are given in **LRFD [Table 6.6.2-1]**. In Wisconsin, use Zone 2 requirements.

Separate fracture toughness requirements are given in **LRFD [Table 6.6.2-2]** for non-fracture-critical and fracture-critical members (or components). A fracture-critical member (FCM) is defined as a component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function. FCMs are subject to more stringent Charpy V-Notch fracture toughness requirements than non-fracture-critical members. For FCMs, High Performance Steel (HPS) shall be used with Zone 2 requirements.

According to **LRFD [6.6.2]**, the engineer has the responsibility to identify all bridge members or components that are fracture critical and clearly delineate their location on the contract plans. Examples of FCMs in bridges include certain truss members in tension, suspension cables, tension components of girders in two-girder systems, pin and link systems in suspended spans, cross girders and welded tie girders in tied-arches. In addition, any attachment having a length in the direction of the tension stress greater than 4 inches and welded to the tension area of a component of a FCM is also to be considered fracture critical.

24.5.3 Non-Redundant Type Structures

Previous AASHTO fatigue and fracture toughness provisions provided satisfactory fracture control for multi-girder structures when employed with good fabrication and inspection practices. However, concern existed that some additional factor of safety against the possibility of brittle fracture should be provided in the design of non-redundant type structures such as single-box and two-box girders, two-plate girders or truss systems where failure of a single element could cause collapse of the structure. A case in point was the collapse of the Point Pleasant Bridge over the Ohio River. HPS shall be used for non-redundant structures.

Primary factors controlling the susceptibility of non-redundant structures to brittle fracture are the material toughness, flaw size and stress level. One of the most effective methods of reducing brittle fracture is lowering the stress range imposed on the member. AASHTO provides an increased safety factor for non-redundant members by requiring a shift of one range of loading cycles for fatigue design with corresponding reduction of stress range for critical stress categories. The restrictive ranges for certain categories require the designer to investigate the use of details which do not fall in critical stress categories or induce brittle fracture. For non-fracture-critical members including bolted tie girders found in tied arch bridges, multiple box girder structures (3 boxes) and hanger plates, HPS shall also be used.

As per a FHWA directive, two-girder box girder structures are to be considered non-redundant.

For I-girder bridges, other than pedestrian or other unusual structures, four or more girders shall be used.

24.6 Design Approach - Steps in Design

24.6.1 Obtain Design Criteria

The first design step for a steel girder is to choose the correct design criteria. The design criteria include the following:

- Number of spans
- Span lengths
- Skew angles
- Number of girders
- Girder spacing
- Deck overhang
- Cross-frame spacing
- Flange and web yield strengths
- Deck concrete strength
- Deck reinforcement strength
- Deck thickness
- Dead loads
- Roadway geometry
- Haunch depth

For steel girder design, the following load combinations are generally considered:

- Strength I
- Service II
- Fatigue I

The extreme event limit state (including earthquake load) is generally not considered for a steel girder design.

The following steps are taken in determining the girder or beam spacing and the slab thickness:

1. The girder spacing (and the resulting number of girders) for a structure is determined by considering the desirable girder depth and the span lengths. Refer to 24.4.2 for design aids. Where depth or deflection limitations do not control the design, it is usually more economical to use fewer girders with a wider spacing and a thicker slab. Four girders are generally considered to be the minimum, and five girders are desirable to facilitate future redecking.

2. The slab overhang on exterior girders is limited to 3'-7" measured from the girder centerline to the edge of slab. The overhang is limited to prevent rotation and bending of the web during construction caused by the forming brackets. The overhang width is generally determined such that the moments and shears in the exterior girder are similar to those in the interior girder. In addition, the overhang is set such that the positive and negative moments in the deck slab are balanced. A common rule of thumb is to make the overhang approximately 0.28 to 0.5 times the girder spacing.

3. Check if a thinner slab and the same number of members can be used by slightly reducing the spacing.

24.6.2 Select Trial Girder Section

Before the dead load effects can be computed, a trial girder section must be selected. This trial girder section is selected based on previous experience and based on preliminary design. Based on this trial girder section, section properties and dead load effects will be computed. Then specification checks will be performed to determine if the trial girder section successfully resists the applied loads. If the trial girder section does not pass all specification checks or if the girder optimization is not acceptable, then a new trial girder section must be selected and the design process must be repeated.

The following tips are presented to help bridge designers in developing an economical steel girder for most steel girder designs. Other design tips are available in various publications from the American Institute of Steel Construction (AISC) and from steel fabricators.

- Girder depth – The minimum girder depth is specified in LRFD [2.5.2.6.3]. An estimate of the optimum girder depth can be obtained from trial runs using design software. The web depth may be varied by several inches more or less than the optimum without significant cost penalty. Refer to 24.4.2 for recommended girder depths for a given girder spacing and span length.

- Web thickness – A "nominally stiffened" web (approximately 1/16 inch thinner than "unstiffened") will generally provide the least cost alternative or very close to it. However, for web depths of approximately 50” or less, unstiffened webs may be more economical.

- Plate transitions – For rolled sections, a change in section should occur only at field splice locations. For plate girders, include the change in section at butt splices and check the maximum rolling lengths of plates to see if additional butt splices are required. The fabricator may assume the cost of extending the heavier plate and eliminating the butt splice; this option has been used by fabricators on numerous occasions. Shim plates are provided at the bearing to allow for either option. A common rule of thumb is to use no more than three plates (two shop splices) in the top or bottom flange of field sections up to 130 feet long. In some cases, a single flange plate size can be carried through the full length of the field section. Estimate field splice locations at approximately the 7/10 point of continuous spans.

- Flange widths – Flange widths should remain constant within field sections. The use of constant flange widths simplifies construction of the deck. The unsupported length in compression of the shipping piece divided by the minimum width of the compression flange in that piece should be less than approximately 85. High bearing reactions at the piers of continuous girders may govern the width of the bottom flange.

- Flange transitions – It is good design practice to reduce the flange cross-sectional area by no more than approximately one-half of the area of the heavier flange plate. This reduces the build-up of stress at the transition.

- Haunched girders – On haunched plate girders, the length of the parabolic haunch is approximately 1/4 of the span length. The haunch depth is 1 1/2 times the midspan depth.

It should be noted that during the optimization process, minor adjustments can be made to the plate sizes and transition locations without needing to recompute the analysis results. However, if significant adjustments are made, such that the moments and shears would change significantly, then a revised analysis is required.

24.6.3 Compute Section Properties

See 17.2.11 for determining composite slab width.

For a composite superstructure, several sets of section properties must be computed. The initial dead loads (or the non-composite dead loads) are applied to the girder-only section. The superimposed dead loads are applied to the composite section based on a modular ratio of 3n, as described in **LRFD [6.10.1.1.1]**. The live loads are applied to the composite section based on a modular ratio of n.

For girders with shear connectors provided throughout their entire length and with slab reinforcement satisfying the provisions of **LRFD [6.10.1.7]**, stresses due to loads applied to the composite section for the Fatigue I and Service II limit states may be computed using the short-term composite section, based on a modular ratio of n, assuming the concrete slab to be fully effective for both positive and negative flexure.

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not.

For LRFD, Wisconsin places shear connectors in both the positive and negative moment regions of continuous steel girder bridges, and both regions are considered composite in analysis and design computations. Negative flexure concrete deck reinforcement is considered in the section property calculations.

24.6.4 Compute Dead Load Effects

The girder must be designed to resist the dead load effects, as well as the other load effects. Various types of dead loads and their corresponding load factors are described in 17.2.4 and 17.2.5.

For the steel girder, the dead load per unit length varies due to the change in plate sizes. The moments and shears due to the weight of the steel girder can be computed using analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

Distribution of dead load to the girders is described in 17.2.8.

The stiffness of the composite section is used for determining live load and composite dead load moments and shears. When computing live load values, the composite section is based on n, and when computing composite dead load values, the composite section is based on 3n. Non-composite dead load moments and shears are computed based on the stiffness of the non-composite steel section.

24.6.5 Compute Live Load Effects

The girder must also be designed to resist the live load effects. The live load consists of an HL-93 loading. Similar to the dead load, the live load moments and shears for an HL-93 loading can be obtained from an analysis computer program.

For all limit states other than fatigue and fracture, the dynamic load allowance, IM, is 0.33. However, it is important to note that the dynamic load allowance is applied only to the design truck or tandem. The dynamic load allowance is not applied to pedestrian loads or to the design lane load.

Live load distribution factors must be computed as specified in **LRFD [4.6.2.2]**, as shown in Table 24.6-1.

WisDOT Policy Item

For beams with variable moment of inertia, the longitudinal stiffness parameter, Kg (**LRFD [4.6.2.2.1-1]**), shall be based on a weighted average of properties, over the entire length of the bridge.

In addition to computing the live load distribution factors, their ranges of applicability must also be checked. If they are not satisfied, then conservative assumptions must be made based on sound engineering judgment. Additional information about distribution of live load to the girders is presented in 17.2.8.

For skewed bridges, WisDOT does not consider skew correction factors for moment.

Live Load Distribution Factor	AASHTO LRFD Reference
Moments in Interior Beams	**LRFD [Table 4.6.2.2.2b-1]**
Moments in Exterior Beams	**LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.2d-1]**
Moment Reduction for Skew	Not Applicable for WisDOT
Shear in Interior Beams	**LRFD [Table 4.6.2.2.3a-1]**
Shear in Exterior Beams	**LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.3b-1]**
Shear Correction for Skew	**LRFD [Table 4.6.2.2.3c-1]**

Table 24.6-1
Live Load Distribution Factors

24.6.6 Combine Load Effects

The next step is to combine the load effects for each of the applicable limit states. Load effects are combined in accordance with **LRFD [Table 3.4.1-1]** and **LRFD [Table 3.4.1-2]**.

After combining load effects, the next ten design steps consist of verifying the structural adequacy of the steel girder using appropriate sections of *AASHTO LRFD*. For steel girder designs, specification checks are generally performed at the following locations:

- Span tenth points
- Locations of plate transitions
- Locations of stiffener spacing transitions

However, it should be noted that the maximum moment within a span may not necessarily occur at any of the above locations.

Check the loads of the interior and exterior members to see if one or both members are to be designed.

24.6.7 Check Section Property Limits

Several checks are required to ensure that the proportions of the girder section are within specified limits, as presented in **LRFD [6.10.2]**. The first section proportion check relates to the web slenderness, and the second set of section proportion checks relate to the general proportions of the section.

24.6.8 Compute Plastic Moment Capacity

For composite sections, the plastic moment, M_p, must be calculated as the first moment of plastic forces about the plastic neutral axis. The methodology for the plastic moment capacity computations is presented in **LRFD [Appendix D6.1]**.

24.6.9 Determine If Section is Compact or Non-compact

The next step in the design process is to determine if the section is compact or non-compact, as described in **LRFD [6.10.6.2.2]**. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

24.6.10 Design for Flexure – Strength Limit State

The next step is to compute the flexural resistance of the girder at each section. These computations vary, depending on whether the section is composite or non-composite, whether the section is compact or non-compact, and whether the section is in positive flexure or negative flexure. The following sections of *AASHTO LRFD* can be used:

- Compact, composite section in positive flexure – **LRFD [6.10.7.1]**
- Non-compact, composite section in positive flexure – **LRFD [6.10.7.2]**
- Composite sections in negative flexure – **LRFD [6.10.8]**
- Non-composite sections – **LRFD [6.10.8]**

WisDOT Policy Item:

Do not utilize optional **LRFD [Appendix B6]** for Moment Redistribution from Interior-Pier I-Sections in Straight Continuous-Span Bridges.

24.6.11 Design for Shear

Shear must be checked at each section of the girder. However, shear is generally maximum at or near the supports.

The first step in the design for shear is to check if the web must be stiffened. A "nominally stiffened" web (approximately 1/16 inch thinner than "unstiffened") will generally provide the least cost alternative or very close to it. However, for web depths of approximately 50 inches or less, unstiffened webs may be more economical.

It should be noted that in end panels, the shear is limited to either the shear yield or shear buckling in order to provide an anchor for the tension field in adjacent interior panels. Tension field is not allowed in end panels. The design procedure for shear in the end panel is presented in **LRFD [6.10.9.3.3]**.

24.6.12 Design Transverse Intermediate Stiffeners and/or Longitudinal Stiffeners

If transverse intermediate stiffeners and/or longitudinal stiffeners are used, they must be designed. The design of transverse intermediate stiffeners is described in 24.10, and the design of longitudinal stiffeners is described in 24.11.

24.6.13 Design for Flexure – Fatigue and Fracture

Load-induced fatigue must be considered in a steel girder design. Fatigue considerations may include:

- Welds connecting the shear studs to the girder
- Welds connecting the flanges and the web
- Welds connecting stiffeners to the girder

The specific fatigue considerations depend on the unique characteristics of the girder design. Specific fatigue details and detail categories are explained and illustrated in **LRFD [Table 6.6.1.2.3-1]**.

In addition to the nominal fatigue resistance computations, fatigue requirements for webs must also be checked. These checks are required to control out-of-plane flexing of the web due to flexure or shear under repeated live loading.

24.6.14 Design for Flexure – Service Limit State

The girder must be checked for service limit state control of permanent deflection. This check is intended to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. Service II is used for this check.

In addition to the check for service limit state control of permanent deflection, the girder must also be checked for live load deflection, as described in 24.4.3.

24.6.15 Design for Flexure – Constructability Check

The girder must also be checked for flexure during construction. The girder has already been checked in its final condition when it behaves as a composite section. It is the responsibility of the contractor to ensure that allowable stresses aren't exceeded during steel erection. The engineer is to make certain allowable stresses aren't exceeded from the time the steel erection is complete through final service, including during the deck pour. In addition, check the lateral bracing without the deck slab.

Before constructability checks can be performed, the slab pouring sequence must be determined. Refer to Standard for Slab Pouring Sequence. Determine the maximum amount of concrete that can be poured in a day. Determine deflections based on the proposed pouring sequence. The effects of the deck pouring sequence will often control the design of the top flange in the positive moment regions of composite girders.

Lateral torsional buckling can occur when the compression flange is not laterally supported. The laterally unsupported compression flange tends to buckle out-of-plane between the points of lateral support. Because the tension flange is kept in line, the girder section twists when it moves laterally. This behavior is commonly referred to as lateral torsional buckling. Lateral torsional buckling is generally most critical for the moments induced during the deck pouring sequence. If lateral torsional buckling occurs, the plastic moment resistance, Mp, cannot be reached.

In addition to checking the nominal flexural resistance during construction, the nominal shear resistance must also be checked.

24.6.16 Check Wind Effects on Girder Flanges

The next step is to check wind effects on the girder flanges. Wind effects generally do not control a steel girder design, and they are generally considered for the exterior girders only.

24.6.17 Draw Schematic of Final Steel Girder Design

If all of the above specification checks are satisfied, then the trial girder section is acceptable and can be considered the final girder section. It is often useful to draw a schematic summarizing the design of the final girder section.

However, if any of the specification checks are not satisfied or if the design is found to be overly conservative, then the trial girder section must be revised appropriately, and the specification checks must be repeated for the new trial girder section.

24.6.18 Design Bolted Field Splices

If bolted field splices are used, they must be designed, as described in 24.8.

24.6.19 Design Shear Connectors

For a composite steel girder, the shear connectors must be designed, as described in 24.7.5. The shear connector spacing must be computed based on fatigue and strength limit states.

24.6.20 Design Bearing Stiffeners

The next step is to design the bearing stiffeners, as described in 24.9.

24.6.21 Design Welded Connections

Welded connections are required at several locations on the steel superstructure, and all welds must be designed. Base metal, weld metal and welding design details must conform to the requirements of the *ANSI/AASHTO/AWS Bridge Welding Code D1.5*.

In most cases, the minimum weld thickness provides a welded connection that satisfies all design requirements. Therefore, the minimum weld thickness is generally a good starting point when designing a fillet weld.

Determine the distance from the piers where field welding to the top flange for construction purposes is not permitted (other than for shear studs).

24.6.22 Design Diaphragms, Cross-Frames and Lateral Bracing

Diaphragms and cross-frames must be designed in accordance with **LRFD [6.7.4]**. Diaphragms and cross-frames may be placed at the following locations along the bridge:

- At the end of the structure
- Across interior supports
- Intermittently along the span

When investigating the need for diaphragms or cross-frames and when designing them, the following must be considered:

- Transfer of lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings
- Stability of the bottom flange for all loads when it is in compression
- Stability of the top flange in compression prior to curing of the deck
- Distribution of vertical dead and live loads applied to the structure

Diaphragms or cross-frames can be specified as either temporary (if they are required only during construction) or permanent (if they are required during construction and in the bridge's final condition).

At a minimum, *AASHTO LRFD* requires that diaphragms and cross-frames be designed for the following transfer of wind loads based on **LRFD [4.6.2.7]** and for applicable slenderness requirements in accordance with **LRFD [6.8.4]** or **LRFD [6.9.3]**. In addition, connection plates must satisfy the requirements of **LRFD [6.6.1.3.1]**.

Refer to Standards 24.03 through 24.06 for information about the design of lateral bracing and end diaphragms. Consideration must be given to connection details susceptible to fatigue crack growth.

24.6.23 Determine Deflections, Camber, and Elevations

Determine the dead load deflections, blocking, camber, top of steel elevations and top of slab elevations. Camber and blocking are described in 24.4.8.

24.7 Composite Design

24.7.1 Composite Action

Composite action is present in steel girder superstructures when the steel beams or girders feature shear connectors which are embedded within the concrete slab. The shear connectors prevent slip and vertical separation between the bottom of the slab and the top of the steel member. Unless temporary shoring is used, the steel members deflect under the dead load of the wet concrete before the shear connectors become effective. However, since temporary shoring is not used in Wisconsin, composite action applies only to live loads and to portions of dead load placed after the concrete deck has hardened.

In the positive moment region, the concrete deck acts in compression and the composite section includes the slab concrete. However, in the negative moment region, the concrete deck acts in tension and the composite section includes the bar steel reinforcement in the slab.

As previously described, for LRFD, Wisconsin places shear connectors in both the positive and negative moment regions of continuous steel girder bridges, and both regions are considered composite in analysis and design computations. Negative flexure concrete deck reinforcement is considered in the section property calculations.

WisDOT policy item:

For rehabilitation projects, do not add shear studs in the negative moment region if none exist. Likewise, do not add additional studs in the positive moment region if shear connectors are provided and were designed for shear (not slab anchors on approximately 3'-0" to 4'-0" spacing).

If slab anchors are provided, consider as non-composite and add shear connectors if necessary for rating purposes. If adequate shear connector embedment into the deck is not achieved, additional reinforcement should be provided as per Figure 17.5-1.

24.7.2 Values of n for Composite Design

The effective composite concrete slab is converted to an equivalent steel area by dividing by n. For f'_c = 4 ksi, use n = 8.

f'_c = Minimum ultimate compressive strength of the concrete slab at 28 days

n = Ratio of modulus of elasticity of steel to that of concrete

The actual calculation of creep stresses in composite girders is theoretically complex and not necessary for the design of composite girders. Instead, a simple approach has been adopted for design in which a modular ratio appropriate to the duration of the load is used to compute the corresponding elastic section properties. As specified in **LRFD [6.10.1.1.1b]**, for transient loads applied to the composite section, the so-called ″short-term″ modular ratio, n, is used. However, for permanent loads applied to the composite section, the so-called ″long-term″ modular ratio, 3n, is used. The short-term modular ratio is based on the initial tangent modulus, E_c, of the concrete, while the long-term modular ratio is based on an effective apparent modulus, E_c/k, to account for the effects of creep. In U.S. practice, a value of k equal to 3 has been accepted as a reasonable value.

24.7.3 Composite Section Properties

The minimum effective slab thickness is equal to the nominal slab thickness minus 1/2" for wearing surface. The maximum effective slab width is defined in **LRFD [4.6.2.6]**.

24.7.4 Computation of Stresses

24.7.4.1 Non-composite Stresses

For non-composite sections, flexural stresses are computed using only non-composite (steel-only) section properties, as follows:

$$f_b = \frac{DLM\,(DC1)}{S(\text{steel only})} + \frac{DLM\,(DC2\ \&\ DW)}{S(\text{steel only})} + \frac{LLM\,(\text{Traffic})}{S(\text{steel only})} + \frac{LLM\,(\text{Pedestrian})}{S(\text{steel only})}$$

24.7.4.2 Composite Stresses

For composite sections, flexural stresses in the steel girder subjected to positive flexure are computed using appropriate non-composite (steel-only) and composite section properties, as follows:

$$f_b = \frac{DLM\,(DC1)}{S(\text{steel only})} + \frac{DLM\,(DC2\ \&\ DW)}{S(\text{composite},3n)} + \frac{LLM\,(\text{Traffic})}{S(\text{composite},n)} + \frac{LLM\,(\text{Pedestrian})}{S(\text{composite},n)}$$

For composite sections, flexural stresses in the concrete deck subjected to positive flexure are computed as follows:

$$f_b = \frac{DLM(DC2 + DW)}{S(\text{composite},n)} + \frac{LLM(\text{Traffic})}{S(\text{composite},n)} + \frac{LLM(\text{Pedestrian})}{S(\text{Composite},n)}$$

Where:

f_b	=	Computed steel flexural stress
DLM	=	Dead lead moment
LLM	=	Live load moment
S	=	Elastic section modulus
DC1	=	DC dead load resisted by the steel section only (for example, steel girder, concrete deck, concrete haunch, cross-frames and stiffeners)

DC2 = DC dead load resisted by the composite section (for example, concrete parapets)

DW = Dead load due to future wearing surface and utilities

24.7.5 Shear Connectors

Refer to Standard for Plate Girder Details for shear connector details. Three shop or field welded 7/8" diameter studs at a length of 5" are placed on the top flange. The studs are equally spaced with a minimum clearance of 1 1/2" from the edge of the flange. On girders having thicker haunches where stud embedment is less than 2" into the slab, longer studs should be used to obtain the minimum embedment of 2".

Connectors which fall on the flange field splice plates should be repositioned near the ends of the splice plate. The maximum spacing of shear connectors is 2'. Connector spacings should begin a minimum of 9" from the centerline of abutments.

To determine the locations of shear connectors along the length of the girder, two general requirements must be satisfied:

- Spacing (or pitch) requirements governed by fatigue, as presented in **LRFD [6.10.10.1]**
- Number of connector requirements governed by strength, as presented in **LRFD [6.10.10.4]**

For the fatigue limit state, the pitch, p, of the shear connectors must satisfy the following equation:

$$p \leq \frac{nZ_r}{V_{sr}}$$

Where:

N = Number of shear connectors in a cross section

V_{sr} = Horizontal fatigue shear range per unit length (kips/in.)

Z_r = Shear fatigue resistance of an individual shear connector determined as specified in **LRFD [6.10.10.2]** (kips)

When computing the value for V_{sr}, the maximum value of composite moment of inertia in the span can be used.

For the strength limit state, the minimum number of required shear connectors, n, is computed for a given region according to the following equation:

$$n = \frac{P}{Q_r}$$

Where:

P = Total nominal shear force determined as specified in **LRFD [6.10.10.4.2]** (kips)

Q_r = Factored shear resistance of one shear connector (kips)

The given regions over which the required number of shear connectors is distributed are defined based on the point of maximum moment due to live load plus dynamic load allowance. This value is used because it applies to the composite section and is easier to locate than a maximum of the sum of all the moments acting on the composite section.

In most cases, the connector spacing (using three connectors per row) based on fatigue requirements is more than adequate for the strength design requirements. However for relatively long spans, additional shear connectors may be required to satisfy the strength design requirements.

In addition to the above general requirements, special shear connector requirements at points of permanent load contraflexure are presented in **LRFD [6.10.10.3]**.

Additional information and equations used for LRFD design of shear connectors are presented in **LRFD [6.10.10]**. In addition, a design example for shear connectors is also provided in this *Bridge Manual*.

24.7.6 Continuity Reinforcement

For continuous steel girder bridges, continuity reinforcement in the concrete deck must be considered in regions of negative flexure, as specified in **LRFD [6.10.1.7]**. Continuity reinforcement consisting of small bars with close spacing is intended to control concrete deck cracking.

If the longitudinal tensile stress in the concrete deck due to either the factored construction loads or the Service II load combination exceeds ϕf_r, then the following continuity reinforcement requirements must be satisfied:

- The total cross-sectional area of the longitudinal reinforcement in the deck shall be greater than or equal to one percent of the total cross-sectional area of the concrete deck.
- The required reinforcement shall be placed in two layers uniformly distributed across the deck width, with two-thirds being in the top layer and one-third in the bottom layer.

- The specified minimum yield strength, fy, of the reinforcing steel shall not be less than 60 ksi.
- The size of the reinforcement bars shall not exceed No. 6 bars.
- The spacing of the reinforcement bars shall not exceed 12 inches.

Tables 17.5-3 and 17.5-4 meet the criteria specified above.

In computing ϕf_r, f_r shall be taken as the modulus of rupture of the concrete (see **LRFD [5.4.2.6]**) and ϕ shall be taken as 0.90, which is the appropriate resistance factor for concrete in tension (see **LRFD [5.5.4.2.1]**). The longitudinal stresses in the concrete deck are computed as specified in **LRFD [6.10.1.1.1d]**. Superimposed dead loads and live loads are considered to be resisted by the composite section using the short-term modular ratio, n. Non-composite dead loads are supported by the girders alone.

Terminate the continuity reinforcement at the point of non-composite dead load contraflexure plus a development length. The bars are lapped to No. 4 bars.

For non-composite slabs in the negative moment region (on rehabilitation projects), extend the longitudinal reinforcement in Tables 17.5-3 and 17.5-4 a development length beyond the first shear connectors.

24.8 Field Splices

24.8.1 Location of Field Splices

Field splices shall be placed at the following locations whenever it is practical:

- At or near a point of dead load contraflexure for continuous spans
- Such that the maximum rolling length of the flange plates is not exceeded, thus eliminating one or more butt splices
- At a point where the fatigue in the net section of the base metal is minimized
- Such that section lengths between splices are limited to 120', unless special conditions govern

24.8.2 Splice Material

For homogeneous girders, the splice material is the same as the members being spliced. Generally, 3/4" diameter high strength A325 bolted friction-type connectors are used unless the proportions of the structure warrant larger diameter bolts.

24.8.3 Design

The following is a general description of the basic steps required for field splice design. These procedures and the accompanying equations are described in greater detail in **LRFD [6.13.6]**.

24.8.3.1 Obtain Design Criteria

The first design step is to identify the appropriate design criteria. This includes defining material properties, identifying relevant superstructure information and determining the splice location based on the criteria presented in 24.8.1.

Resistance factors used for field splice design are as presented in 17.2.6.

When calculating the nominal slip resistance of a bolt in a slip-critical connection, the value of the surface condition factor, K_s, shall be taken as follows:

- For structures that are to be painted, use Ks = 0.33.
- For unpainted, blast-cleaned steel or steel with organic zinc paint, use Ks = 0.50.

Where a section changes at a splice, the smaller of the two connected sections should be used in the design, as specified in **LRFD [6.13.6.1.1]**.

24.8.3.1.1 Section Properties Used to Compute Stresses

The section properties used to compute stresses are described in **LRFD [6.10.1.1.1]**.

For calculating flexural stresses in sections subjected to positive flexure, the composite sections for short-term (transient) and long-term (permanent) moments shall be based on n and 3n, respectively.

For calculating flexural stresses in sections subjected to negative flexure, the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck, except as specified otherwise in **LRFD [6.6.1.2.1]**, **LRFD [6.10.1.1.1d]** or **LRFD [6.10.4.2.1]**.

WisDOT policy item:

When computing composite section properties based on the steel section and the longitudinal reinforcement within the effective width of the concrete deck, only the top layer of reinforcement shall be considered.

Where moments due to short-term and long-term loads are of opposite sign at the strength limit state, the associated composite section may be used with each of these moments if the resulting net stress in the concrete deck due to the sum of the factored moments is compressive. Otherwise, the provisions of **LRFD [6.10.1.1.1c]** shall be used to determine the stresses in the steel section. Stresses in the concrete deck shall be determined as specified in **LRFD [6.10.1.1.1d]**.

However, for members with shear connectors provided throughout their entire length that also satisfy the provisions of **LRFD [6.10.1.7]**:

- Flexural stresses caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete deck is effective for both positive and negative flexure, as described in **LRFD [6.10.4.2.1]**.

- Live load stresses and stress ranges for fatigue design may be computed using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure, as described in **LRFD [6.6.1.2.1]**.

WisDOT policy item:

When stresses at the top and bottom of the web are required for web splice design, the flange stresses at the mid-thickness of the flanges can be conservatively used. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations.

24.8.3.1.2 Constructability

As described in **LRFD [6.13.6.1.4a]**, splice connections shall be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck.

24.8.3.2 Compute Flange Splice Design Loads

Commercially available software programs can be used to obtain the design dead loads and live loads at the splice. The live loads should include dynamic load allowance and distribution factors.

Splices are typically designed for the Strength I, Service II and Fatigue I load combinations. The load factors for these load combinations are presented in 17.2.5. The stresses corresponding to these load combinations should be computed at the mid-thickness of the top and bottom flanges.

24.8.3.2.1 Factored Loads

For the Strength I and Service II load combinations, factored loads must be computed for the following two cases:

- Case 1: Dead load + Positive live load
- Case 2: Dead load + Negative live load

For the Fatigue I load combination, the following two load cases are used to compute the factored loads:

- Case 1: Positive live load
- Case 2: Negative live load

Minimum and maximum load factors are applied as appropriate to compute the controlling loading.

24.8.3.2.2 Section Properties

Section properties based on the gross area of the steel girder are used for computation of the maximum flexural stresses due to the factored loads for the Strength I, Service II and Fatigue I load combinations, as described in **LRFD [6.13.6.1.4a]** and **LRFD [C6.13.6.1.4a]**.

24.8.3.2.3 Factored Stresses

After the factored loads and section properties have been computed, factored stresses must be computed for each of the following cases:

- Strength I load combination – Dead load + Positive live load
- Strength I load combination – Dead load + Negative live load
- Service II load combination – Dead load + Positive live load
- Service II load combination – Dead load + Negative live load

- Fatigue I load combination – Positive live load
- Fatigue I load combination – Negative live load

Factored stresses are computed by dividing the factored moments by the appropriate section moduli.

24.8.3.2.4 Controlling Flange

As described in **LRFD [C6.13.6.1.4c]**, the controlling flange is defined as either the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its mid-thickness due to the factored loads for the loading condition under investigation to its factored flexural resistance. The other flange is termed the non-controlling flange. In areas of stress reversal, the splice must be checked independently for both positive and negative flexure. For composite sections in positive flexure, the controlling flange is typically the bottom flange. For sections in negative flexure, either flange may qualify as the controlling flange.

24.8.3.2.5 Flange Splice Design Forces

After the factored stresses have been computed, the flange splice design forces can be computed as specified in **LRFD [6.13.6.1.4c]**. The design forces are computed for both the top and bottom flange for each load case (positive and negative live load). For the Strength I load combination, the design force is computed as the design stress times the smaller effective flange area on either side of the splice. When a flange is in compression, the gross flange area is used.

Service II load combination design forces must also be computed. As specified in **LRFD [6.13.6.1.4c]**, bolted connections for flange splices should be designed as slip-critical connections for the service level flange design force. This design force is computed as the Service II design stress multiplied by the smaller gross flange area on either side of the splice.

The flange slip resistance must exceed the larger of the following:

- Service II flange forces
- Factored flange forces from the moments at the splice due to constructability (erection and/or deck pouring sequence), as described in **LRFD [6.13.6.1.4a]**

For the Fatigue I load combination, the stress range at the mid-thickness of both flanges must be computed.

24.8.3.3 Design Flange Splice Plates

The next step is to design the flange splice plates. The width of the outside plate should be at least as wide as the width of the narrowest flange at the splice. The width of the inside plate must allow sufficient clearance for the web and for inserting and tightening the web and flange splice bolts. Fill plates are used when the flange plate thickness changes at the splice location. A typical flange splice configuration is presented in Figure 24.8-1.

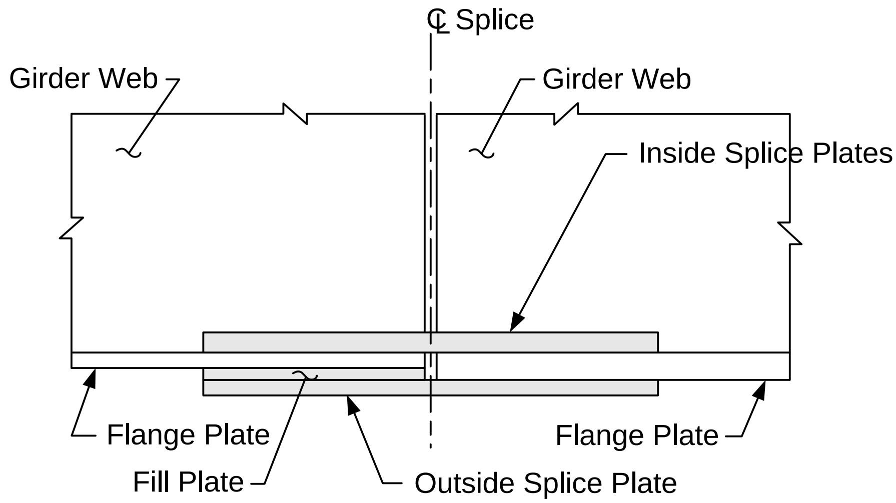

Figure 24.8-1
Bottom Flange Splice Configuration

If the combined area of the inside splice plates is within ten percent of the area of the outside splice plate, then both the inside and outside splice plates may be designed for one-half the flange design force, as described in **LRFD [C6.13.6.1.4c]**. However, if the areas of the inside and outside splice plates differ by more than ten percent, then the flange design force should be proportioned to the inside and outside splice plates. This is calculated by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates.

24.8.3.3.1 Yielding and Fracture of Splice Plates

The design force in the splice plates at the Strength I load combination shall not exceed the factored resistances for yielding and fracture, as described in **LRFD [6.13.5.2]** and **LRFD [6.8.2]**.

For a tension member, the net width shall be determined for each chain of holes extending across the member along any transverse, diagonal or zigzag line. This is determined by subtracting from the width of the element the sum of the width of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain. For non-staggered holes, the minimum net width is the width of the element minus the width of bolt holes in a line straight across the width.

For a compression member, the gross area is used for these design checks.

24.8.3.3.2 Block Shear

All tension connections, including connection plates, splice plates and gusset plates, shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection. Block shear rupture resistance is described in **LRFD [6.13.4]**. A bolt pattern must be assumed prior to checking an assumed block shear failure mode.

Block shear rupture will usually not govern the design of splice plates of typical proportion.

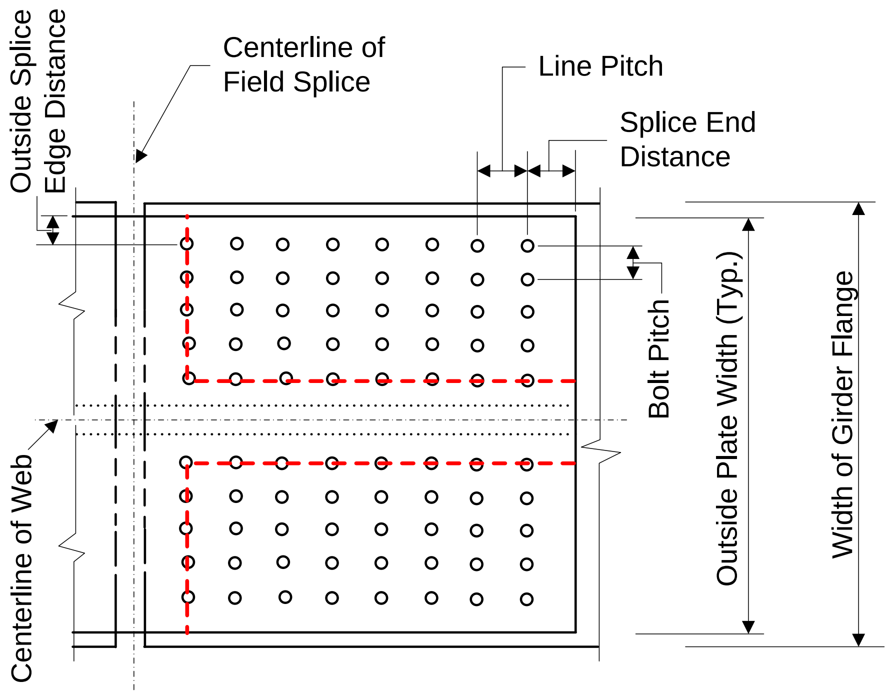

Figure 24.8-2
Double – L Block Shear Path, Flange and Splice Plates

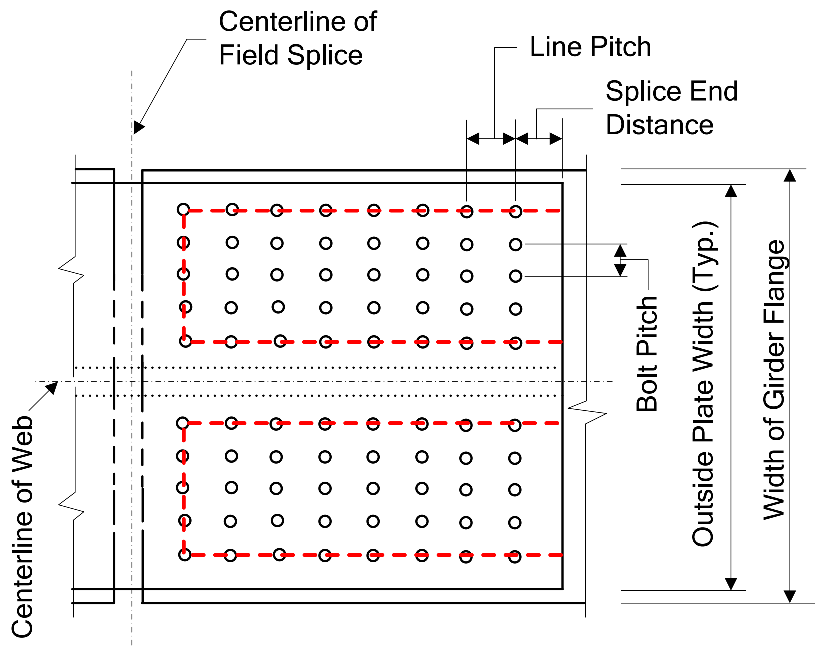

Figure 24.8-3
Double – U Block Shear Path, Flange and Splice Plates

24.8.3.3.3 Net Section Fracture

When checking flexural members at the Strength I load combination or for constructability, all cross sections containing holes in the tension flange must satisfy the fracture requirements of **LRFD [6.10.1.8]**.

24.8.3.3.4 Fatigue of Splice Plates

Check the fatigue stresses in the base metal of the flange splice plates adjacent to the slip-critical connections. Fatigue normally does not govern the design of the splice plates. However, a fatigue check of the splice plates is recommended whenever the area of the flange splice plates is less than the area of the smaller flange to which they are attached.

The fatigue detail category under the condition of Mechanically Fastened Connections for checking the base metal at the gross section of high-strength bolted slip-resistant connections is Category B.

24.8.3.3.5 Control of Permanent Deformation

A check of the flexural stresses in the splice plates at the Service II load combination is not explicitly specified in *AASHTO LRFD*. However, whenever the combined area of the inside and outside flange splice plates is less than the area of the smaller flange at the splice, such a check is recommended.

24.8.3.4 Design Flange Splice Bolts

After the flange splice plates have been designed, the flange splice bolts must be designed for shear, slip resistance, spacing, edge distance and bearing requirements.

24.8.3.4.1 Shear Resistance

Shear resistance computations for bolted connections are described in **LRFD [6.13.2.7]**. The first step is to determine the number of bolts for the flange splice plates that are required to develop the Strength I design force in the flange in shear, assuming the bolts in the connection have slipped and gone into bearing. A minimum of two rows of bolts should be provided to ensure proper alignment and stability of the girder during construction.

The factored resistance of the bolts in shear must be determined, assuming the threads are excluded from the shear planes. For the flange splice bolts, the number of bolts required to provide adequate shear strength is determined by assuming the design force acts on two shear planes, known as double shear.

Requirements for filler plates are presented in **LRFD [6.13.6.1.5]**. When bolts carrying loads pass through fillers 0.25 inches or more in thickness in axially loaded connections, including girder flange splices, either of the following is required:

- The fillers shall be extended beyond the gusset or splice material and shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler.

- The fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the Strength I load combination is reduced by the factor presented in **LRFD [6.13.6.1.5]**.

24.8.3.4.2 Slip Resistance

As specified in **LRFD [6.13.6.1.4c]**, bolted connections for flange splices shall be designed as slip-critical connections for the Service II flange design force or the flange design force from constructability, whichever governs. Slip resistance computations for bolted connections are described in **LRFD [6.13.2.8]**.

When checking for slip of the bolted connection for a flange splice with inner and outer splice plates, the slip resistance should always be determined by dividing the flange design force equally to the two slip planes, regardless of the ratio of the splice plate areas. Slip of the connection cannot occur unless slip occurs on both planes.

24.8.3.4.3 Bolt Spacing

The minimum spacing between centers of bolts in standard holes shall be no less than three times the diameter of the bolt.

The maximum spacing for sealing must be checked to prevent penetration of moisture in the joints, in accordance with **LRFD [6.13.2.6.2]**. Sealing must be checked for a single line adjacent to a free edge of an outside plate or shape (for example, when the bolts along the edges of the plate are parallel to the direction of the applied force) and along the free edge at the end of the splice plate.

24.8.3.4.4 Bolt Edge Distance

Edge distance requirements must be checked as specified in **LRFD [6.13.2.6.6]**. The minimum required edge distance is measured as the distance from the center of any bolt in a standard hole to an edge of the plate.

The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or 5.0 inches.

24.8.3.4.5 Bearing at Bolt Holes

Finally, bearing at the bolt holes must be checked, as specified in **LRFD [6.13.2.9]**. The flange splice bolts are checked for bearing of the bolts on the connected material under the maximum Strength I design force. The design bearing strength of the connected material is calculated as the sum of the bearing strengths of the individual bolt holes parallel to the line of the applied force.

If the bearing resistance is exceeded, it is recommended that the edge distance be increased slightly, in lieu of increasing the number of bolts or thickening the flange splice plates.

24.8.3.5 Compute Web Splice Design Loads

The next step is to compute the web splice design loads for each of the following cases:

- Strength I load combination – Dead load + Positive live load
- Strength I load combination – Dead load + Negative live load
- Service II load combination – Dead load + Positive live load
- Service II load combination – Dead load + Negative live load
- Fatigue I load combination – Positive live load
- Fatigue I load combination – Negative live load

As specified in **LRFD [6.13.6.1.4b]**, web splice plates and their connections shall be designed for the following loads:

- Girder shear forces at the splice location
- Moment due to the eccentricity of the shear at the point of splice

- The portion of the flexural moment assumed to be resisted by the web at the point of the splice

24.8.3.5.1 Girder Shear Forces at the Splice Location

As previously described, any number of commercially available software programs can be used to obtain the design dead loads and live loads at the splice. The live loads must include dynamic load allowance and distribution factors.

24.8.3.5.2 Web Moments and Horizontal Force Resultant

Because the portion of the flexural moment assumed to be resisted by the web is to be applied at the mid-depth of the web, a horizontal design force resultant must also be applied at the mid-depth of the web to maintain equilibrium. The web moment and horizontal force resultant are applied together to yield a combined stress distribution equivalent to the unsymmetrical stress distribution in the web. For sections with equal compressive and tensile stresses at the top and bottom of the web (that is, with the neutral axis located at the mid-depth of the web), the horizontal design force resultant will equal zero.

In the computation of the portion of the flexural moment assumed to be resisted by the web and the horizontal design force resultant in the web, the flange stresses at the midthickness of the flanges can be conservatively used, as described in **LRFD [C6.13.6.1.4b]**. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations.

The moment due to the eccentricity of the design shear is resisted solely by the web and always acts about the mid-depth of the web (that is, the horizontal force resultant is zero). This moment is computed as the design shear times the distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration.

The total web moment for each load case is computed as the sum of these two moments.

In general, the web splice is designed under the conservative assumption that the maximum moment and shear at the splice will occur under the same loading condition.

24.8.3.6 Design Web Splice Plates

After the web splice design forces are computed, the web splice must be designed. First, a preliminary web splice bolt pattern is determined. The outermost rows of bolts in the web splice plate must provide sufficient clearance from the flanges to provide clearance for assembly (see the *AISC Manual of Steel Construction* for required bolt assembly clearances). The web is spliced symmetrically by plates on each side with a thickness not less than one-half the thickness of the web. A typical web splice configuration is presented in Figure 24.8-4.

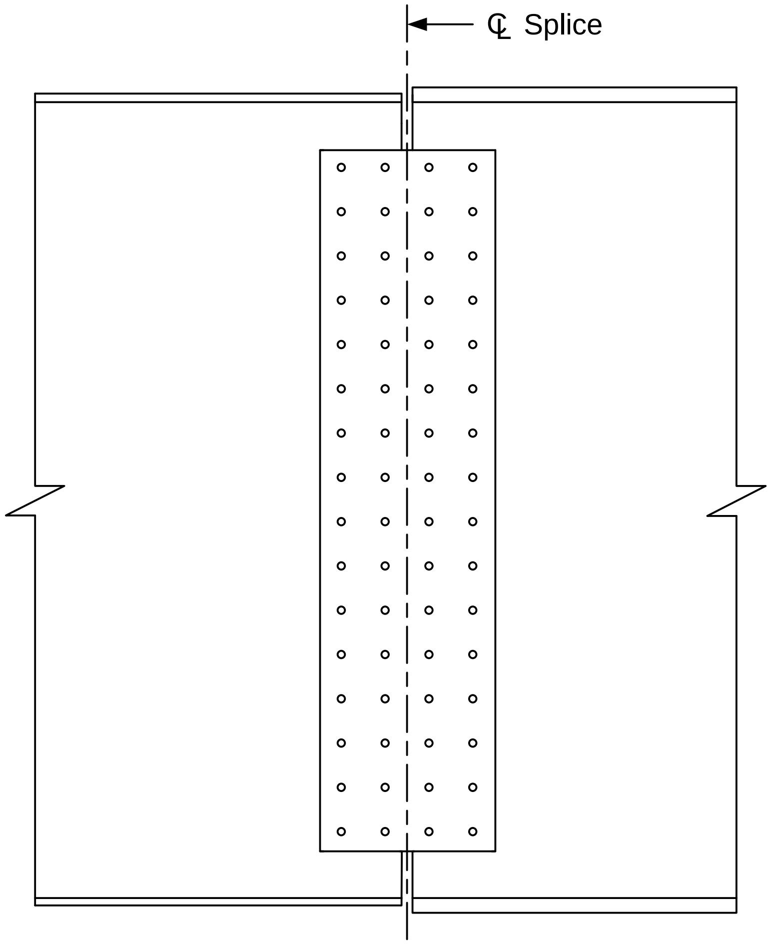

Figure 24.8-4
Web Splice Configuration

The web splice plates should be extended as near as practical the full depth of the web between flanges without impinging on bolt assembly clearances. Also, at least two vertical rows of bolts in the web on each side of the splice should be used. This may result in an over-designed web splice, but it is considered good engineering practice.

24.8.3.6.1 Shear Yielding of Splice Plates

Shear yielding on the gross section of the web splice plates must be checked under the Strength I design shear force, as specified in **LRFD [6.13.6.1.4b]**.

24.8.3.6.2 Fracture and Block Shear Rupture of the Web Splice Plates

Fracture must be investigated on the net section extending across the full plate width, in accordance with **LRFD [6.13.6.1.4b]**. In addition, block shear rupture resistance must be checked in accordance with **LRFD [6.13.4]**. Connection plates, splice plates and gusset plates shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection. Strength I load combination checks for fracture on the net section of web splice plates and block shear rupture normally do not govern for plates of typical proportion.

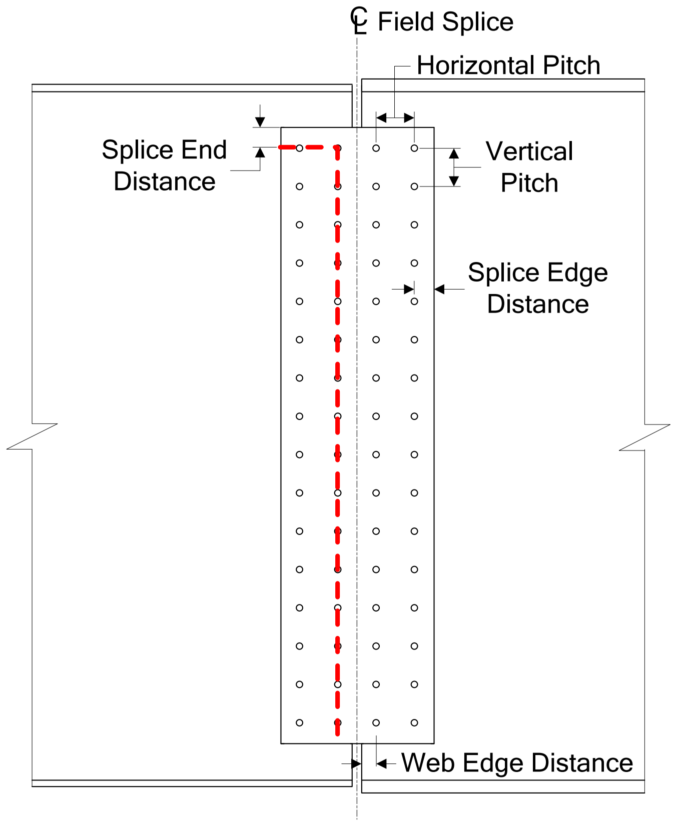

Figure 24.8-5
Block Shear Path, Web Splice

24.8.3.6.3 Flexural Yielding of Splice Plates

Flexural yielding on the gross section of the web splice plates must be checked for the Strength I load combination due to the total web moment and the horizontal force resultant. Flexural yielding must be checked for dead load and positive live load, as well as dead load and negative live load. Flexural yielding of splice plates is checked in accordance with **LRFD [6.13.6.1.4b]**.

24.8.3.6.4 Fatigue of Splice Plates

In addition, fatigue of the splice plates must be checked. Fatigue is checked at the edge of the splice plates which is subject to a net tensile stress. The normal stresses at the edge of the splice plates due to the total positive and negative fatigue load web moments and the corresponding horizontal force resultants are computed.

Check the fatigue stresses in the base metal of the web splice plates adjacent to the slip-critical connections. Fatigue normally does not govern the design of the splice plates. However, a fatigue check of the splice plates is recommended whenever the area of the web splice plates is less than the area of the web at the splice.

The fatigue detail category under the condition of Mechanically Fastened Connections for checking the base metal at the gross section of high-strength bolted slip-resistant connections is Category B.

WisDOT policy item:

For the Fatigue I load combination, the stress range at the mid-thickness of both flanges may be used when checking fatigue in the web.

24.8.3.7 Design Web Splice Bolts

Similar to the flange splice bolts, the web splice bolts must be designed for shear, slip resistance, spacing, edge distance and bearing requirements. These bolt requirements are described in 24.8.3.4.

24.8.3.7.1 Shear in Web Splice Bolts

Shear in the web splice bolts is checked in accordance with **LRFD [6.13.6.1.4b]**. The polar moment of inertia, I_p, of the bolt group on each side of the web centerline with respect to the centroid of the connection is computed as follows:

$$I_p = \frac{n \cdot m}{12} \cdot \left[s^2 \cdot \left(n^2 - 1\right) + g^2 \cdot \left(m^2 - 1\right)\right]$$

Where:

n	=	Number of bolts in each vertical row
m	=	Number of vertical rows of bolts
s	=	Vertical pitch of bolts (inches)
g	=	Horizontal pitch of bolts (inches)

The polar moment of inertia is required to determine the shear force in a given bolt due to the applied web moments. Shear in the web splice bolts is checked for each of the following cases:

- Strength I load combination – Dead load + Positive live load
- Strength I load combination – Dead load + Negative live load
- Service II load combination – Dead load + Positive live load

- Service II load combination – Dead load + Negative live load

Under the most critical combination of the design shear, moment and horizontal force, it is assumed that the bolts in the web splice have slipped and gone into bearing. The shear strength of the bolts are computed assuming double shear and assuming the threads are excluded from the shear planes.

Since the bolt shear strength for both the flange and web splices is based on the assumption that the threads are excluded from the shear planes, an appropriate note should be placed on the drawings to ensure that the splice is detailed to exclude the bolt threads from the shear planes.

24.8.3.7.2 Bearing Resistance at Bolt Holes

Bearing of the web splice bolts on the connected material must be checked for the Strength I load combination, assuming the bolts have slipped and gone into bearing, as specified in **LRFD [6.13.2.9]**. The design bearing strength of the girder web at the location of the extreme bolt in the splice is computed as the minimum resistance along the two orthogonal shear failure planes shown in Figure 24.8-6. The maximum force (vector resultant) acting on the extreme bolt is compared to this calculated strength, which is conservative since the components of this force parallel to the failure surfaces are smaller than the maximum force.

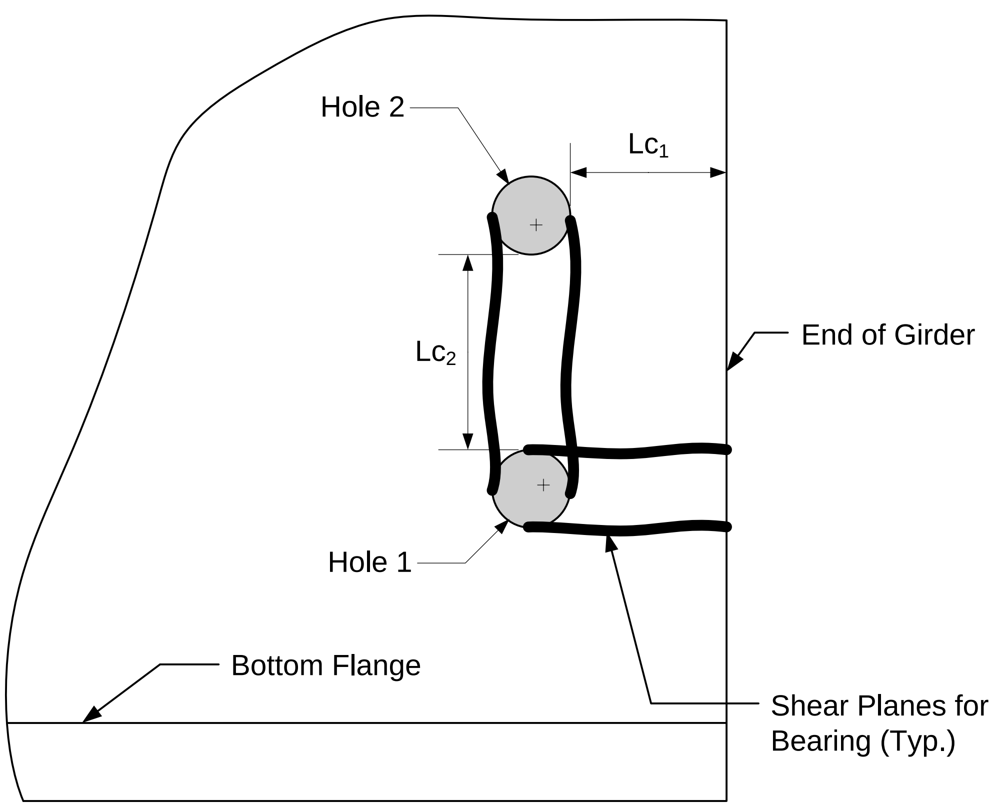

Figure 24.8-6
Bearing Resistance at Girder Web Bolt Holes

To determine the applicable equation for the calculation of the nominal bearing resistance, the clear distance between holes and the clear end distance must be calculated and compared to the value of two times the nominal diameter of the bolt. If the bearing resistance is exceeded, it is recommended that the edge distance be increased slightly, in lieu of increasing the number of bolts or thickening the web splice plates.

24.8.3.8 Schematic of Final Splice Configuration

After the flange splice plates, flange splice bolts, web splice plates and web splice bolts have been designed and detailed, a schematic of the final splice configuration can be developed. A sample schematic of a final splice configuration is presented in Figure 24.8-7.

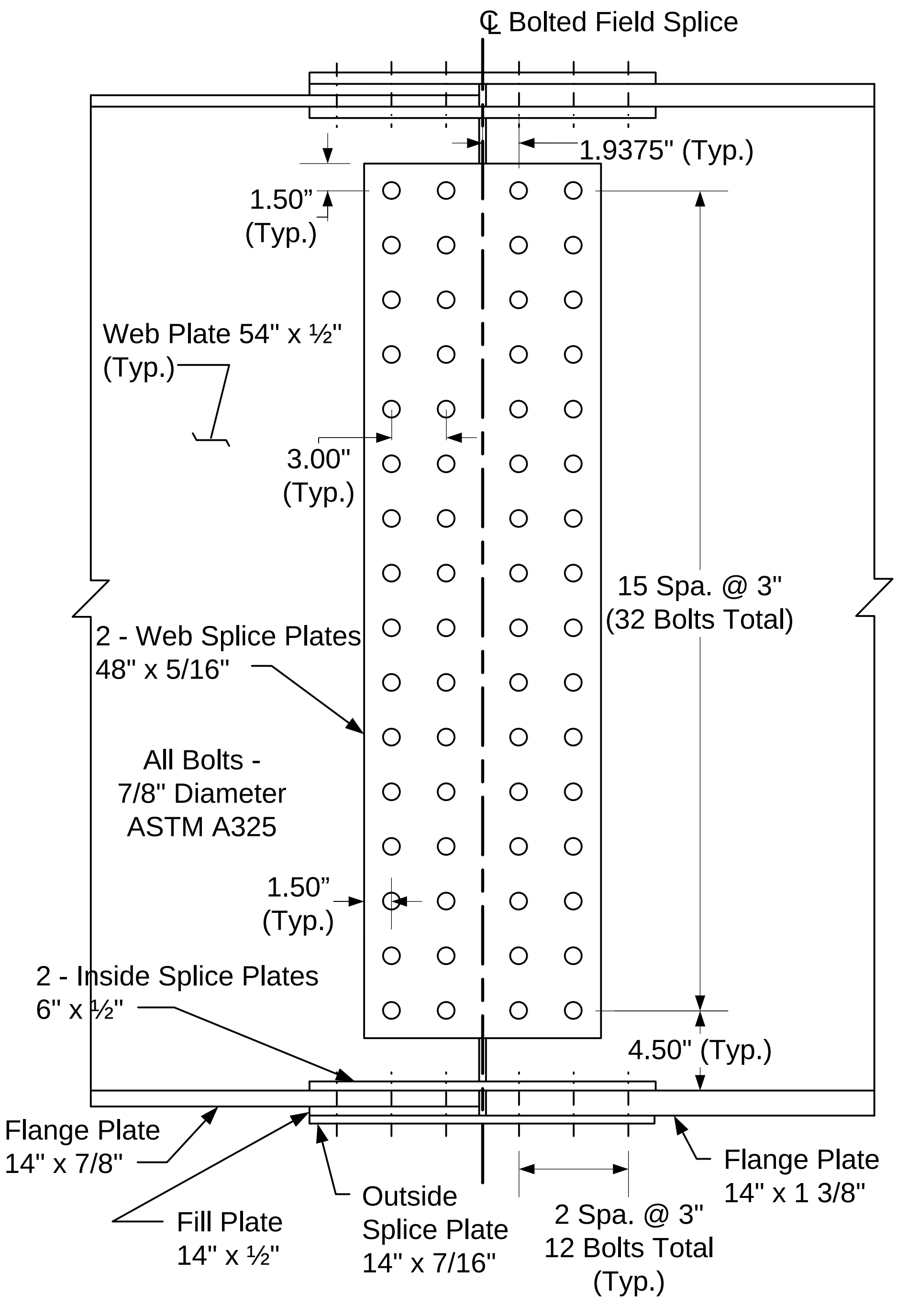

Figure 24.8-7
Sample Schematic of Final Splice Configuration

The schematic includes all plates, dimensions, bolt spacings, edge distances and bolt material and diameter.

A design example for field splices is provided in this *Bridge Manual*.

24.9 Bearing Stiffeners

For skew angles greater than 15°, bearing stiffeners are placed normal to the web of the girder. However, for skew angles of 15° or less, they may be placed parallel to the skew at the abutments and piers to support the end diaphragms or cross framing.

For structures on grades of 3 percent or greater, the end of the girder section at joints is to be cut vertical. This eliminates the large extension and clearance problems at the abutments.

24.9.1 Plate Girders

As specified in **LRFD [6.10.11.2.1]**, bearing stiffeners must be placed on the webs of built-up sections at all bearing locations. Bearing stiffeners are placed over the end bearings of welded plate girders and over the intermediate bearings of continuous welded plate girders. The bearing stiffeners extend as near as practical to the outer edges of the flange plate. They consist of two or more plates placed on both sides of the web. They are ground to a tight fit and fillet welded at the top flange, welded to the web on both sides with the required fillet weld and attached to the bottom flange with full penetration groove welds.

24.9.2 Rolled Beams

At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners must be provided or else the web must satisfy the provisions of **LRFD [D6.5]** (Appendix D to Section 6). According to the provisions of **LRFD [D6.5]**, webs without bearing stiffeners at the indicated locations are to be investigated for the limit states of web local yielding and web crippling. The section must either be modified to comply with these requirements or else bearing stiffeners must be placed on the web at the locations under consideration.

24.9.3 Design

The design of bearing stiffeners is covered in **LRFD [6.10.11.2]**. Bearing stiffeners, which are aligned vertically on the web, are designed as columns to resist the reactions at bearing locations and at other locations subjected to concentrated loads where the loads are not transmitted through a deck or deck system.

24.9.3.1 Projecting Width

As specified in **LRFD [6.10.11.2.2]**, the projecting width, b_t, of each bearing stiffener element must satisfy the following requirement in order to prevent local buckling of the bearing stiffener plates:

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$

Where:

t_p = Thickness of the projecting stiffener element (in.)

E = Modulus of elasticity of stiffener (ksi)

F_{ys} = Specified minimum yield strength of the stiffener (ksi)

The projecting width and thickness of the projecting stiffener element are illustrated in Figure 24.9-1.

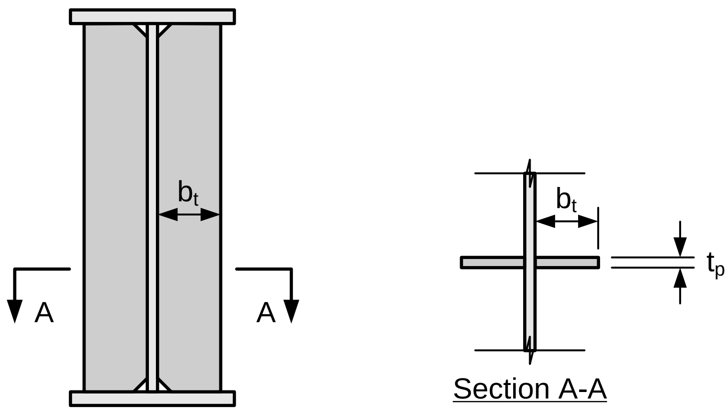

Figure 24.9-1
Projecting Width of a Bearing Stiffener

24.9.3.2 Bearing Resistance

Bearing stiffeners must be clipped to clear the web-to-flange fillet welds and to bring the stiffener plates tight against the flange through which they receive their load. As a result, the area of the plates in direct bearing on the flange is less than the gross area of the plates. As specified in **LRFD [6.10.11.2.3]**, the factored bearing resistance, $(R_{sb})_r$, of the fitted ends of bearing stiffeners is to be taken as:

$$(R_{sb})_r = \phi_b (R_{sb})_n$$

Where:

ϕ_b = Resistance factor for bearing on milled surfaces specified in **LRFD [6.5.4.2]** (= 1.0)

$(R_{sb})_n$ = Nominal bearing resistance for the fitted ends of bearing stiffeners (kips) = 1.4 $A_{pn}F_{ys}$ (**LRFD [6.10.11.2.3-2]**)

A_{pn} = Area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange (in^2)

F_{ys} = Specified minimum yield strength of the stiffener (ksi)

24.9.3.3 Axial Resistance

As previously mentioned, bearing stiffeners are designed as columns. As specified in **LRFD [6.10.11.2.4a]**, the factored axial resistance of the stiffeners, P_r, is to be determined as specified in **LRFD [6.9.2.1]** using the specified minimum yield strength of the stiffener plates, F_{ys}, in order to account for the effect of any early yielding of lower strength stiffener plates. The factored resistance of components in axial compression is given in **LRFD [6.9.2.1]** as:

$$P_r = \phi_c P_n$$

Where:

ϕ_c	=	Resistance factor for axial compression specified in **LRFD [6.5.4.2]** (= 0.90)
P_n	=	Nominal compressive resistance specified in **LRFD [6.9.4.1]** (kips)

For bearing stiffeners, the nominal compressive resistance, P_n, is computed as follows, based on **LRFD [6.9.4.1]**:

If $\lambda \le 2.25$, then: $$P_n = 0.66^{\lambda} F_{ys} A_s$$

If $\lambda > 2.25$, then: $$P_n = \frac{0.88 F_{ys} A_s}{\lambda}$$

Where:

$$\lambda = \left(\frac{Kl}{r_s \pi}\right) \frac{F_{ys}}{E}$$

F_{ys}	=	Specified minimum yield strength of the stiffener (ksi)
A_s	=	Area of effective column section of the bearing stiffeners (see below) (in.^{2})
$K\lambda$	=	Effective length of the effective column taken as 0.75D, where D is the web depth (refer to **LRFD [6.10.11.2.4a]**) (in.)
r_s	=	Radius of gyration of the effective column about the plane of buckling computed about the mid-thickness of the web (refer to **LRFD [6.10.11.2.4a]**) (in.)

24.9.3.4 Effective Column Section

The effective column section of the bearing stiffeners is defined in **LRFD [6.10.11.2.4b]**. For stiffeners bolted to the web, the effective column section is to consist of only the stiffener elements. For stiffeners consisting of two plates welded to the web, the effective column section is to consist of the two stiffener plates, plus a centrally located strip of web extending not more than $9t_w$ on each side of the stiffeners, as illustrated in Figure 24.9-2.

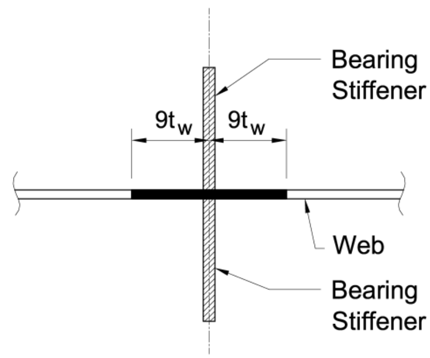

Figure 24.9-2
Effective Column Section for Welded Bearing Stiffener Design (One Pair of Stiffeners)

If more than one pair of stiffeners is used, the effective column section is to consist of all the stiffener plates, plus a centrally located strip of web extending not more than $9t_w$ on each side of the outer projecting elements of the group.

Additional information and equations used for LRFD design of bearing stiffeners are presented in **LRFD [6.10.11.2]**. In addition, a design example for bearing stiffeners is also provided in this *Bridge Manual*.

24.10 Transverse Intermediate Stiffeners

The design of transverse web stiffeners is specified in **LRFD [6.10.11.1]**. Transverse stiffeners are used to increase the shear resistance of a girder and are aligned vertically on the web.

The term connection plate is given to a transverse stiffener to which a cross-frame or diaphragm is connected. A connection plate can serve as a transverse stiffener for shear design calculations.

As specified in **LRFD [6.10.11.1.1]**, stiffeners used as connection plates must be attached to both flanges. According to **LRFD [6.6.1.3.1]**, attachment of the connection plate to the flanges must be made by welding or bolting. When the diaphragms are connected to the transverse intermediate stiffeners, the stiffeners are welded to both the tension and compression flanges. Flange stresses are usually less than the Category C allowable fatigue stresses produced by this detail which the designer should verify.

Stiffeners in straight girders not used as connection plates are to be welded to the compression flange and tight fit to the tension flange. A tight fit can help straighten the flange tilt without the application of heat. According to **LRFD [6.10.11.1.1]**, single-sided stiffeners on horizontally curved girders should be attached to both flanges to help retain the cross-sectional shape of the girder when subjected to torsion and to avoid high localized bending within the web, particularly near the top flange due to the torsional restraint of the concrete deck. For the same reason, it is required that pairs of transverse stiffeners on horizontally curved girders be tight fit or attached to both flanges.

Indicate on the plans the flange to which stiffeners are welded. The stiffeners are attached to the web with a continuous fillet weld. The dead load moment diagram is used to define the compression flange.

In the fabrication of tub sections, webs are often joined to top flanges and the connection plates and transverse stiffeners (not serving as connection plates) are installed, and then these assemblies are attached to a common box flange. The details in this case must allow the welding head to clear the bottom of the connection plates and stiffeners so the webs can be welded continuously to the box flange inside the tub section. A detail must also be provided to permit the subsequent attachment of the connection plates to the box flange (and any other transverse stiffeners that are to be attached to the box flange).

In Wisconsin, if longitudinal stiffeners are required, the transverse stiffeners are placed on one side of the web of the interior member and the longitudinal stiffener on the opposite side of the web. Place intermediate stiffeners on one side of interior members when longitudinal stiffeners are not required. Transverse stiffeners are placed on the inside web face of exterior members. If longitudinal stiffeners are required, they are placed on the outside web face of exterior members as shown on Standard for Plate Girder Details.

Transverse stiffeners can be eliminated by increasing the thickness of the web. On plate girders under 50" in depth, consider thickening the web to eliminate all transverse stiffeners. Within the constant depth portion of haunched plate girders over 50" deep, consider thickening the web to eliminate the longitudinal stiffener and most, but likely not all, of the transverse stiffeners within the span. The minimum size of transverse stiffeners is 5 x ½".

Transverse stiffeners are placed on the inside face of all exterior girders where the slab overhang exceeds 1'-6" as shown on Standard for Plate Girder Details. The stiffeners are to prevent web bending caused by construction of the deck slab where triangular overhang brackets are used to support the falsework.

If slab overhang is allowed to exceed the recommended 3'-7" on exterior girders, the web and stiffeners should be analyzed to resist the additional bending during construction of the deck. Overhang construction brackets may overstress the stiffeners. It may also be necessary to provide longitudinal bracing between stiffeners to prevent localized web deformations which did occur on a structure having 5' overhangs.

24.10.1 Proportions

As specified in **LRFD [6.10.11.1.2]**, the width, b_t, of each projecting transverse stiffener element must satisfy requirements related to the web depth, the flange width and the thickness of the projecting stiffener elements. The width, b_t, is illustrated in Figure 24.10-1.

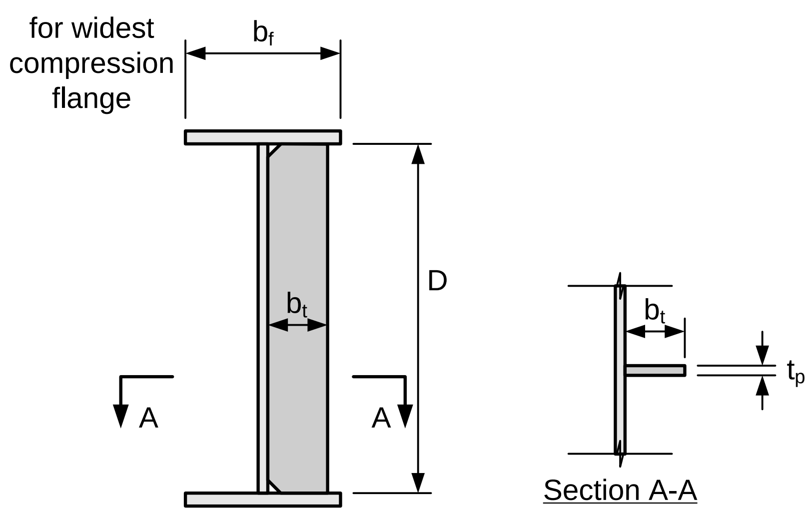

Figure 24.10-1
Projecting Width of Transverse Stiffeners

Fabricators generally prefer a ½" minimum thickness for stiffeners and connection plates.

24.10.2 Moment of Inertia

For the web to adequately develop the shear-buckling resistance, or the combined shear-buckling and post-buckling tension-field resistance, the transverse stiffener must have sufficient rigidity to maintain a vertical line of near zero lateral deflection of the web along the line of the stiffener. Therefore, the bending rigidity (or moment of inertia) is the dominant parameter governing the performance of transverse stiffeners.

As specified in **LRFD [6.10.11.1.3]**, for transverse stiffeners adjacent to web panels in which neither panel supports shear forces larger than the shear-buckling resistance, the moment of inertia of the transverse stiffener, I_t, must satisfy the smaller of the following two equations:

$$I_t \geq b t_w^3 J$$

and

$$I_t \geq \frac{D^4 \rho_t^{1.3}}{40} \left(\frac{F_{yw}}{E} \right)^{1.5}$$

Where:

I_t	=	Moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (in.^{4})
B	=	Smaller of d_o and D (in.)
d_o	=	Smaller of the adjacent panel widths (in.)
D	=	Web depth (in.)
t_w	=	Web thickness (in.)
J	=	Stiffener bending rigidity parameter taken as follows:

$$J = \frac{2.5}{\left(d_o / D \right)^2} - 2.0 \geq 0.5$$

ρ_t	=	Larger of F_{yw}/F_{crs} and 1.0
F_{yw}	=	Specified minimum yield strength of the web (ksi)
F_{crs}	=	Local buckling stress for the stiffener (ksi) taken as follows:

$$Fcrs = \frac{0.31E}{\left(b_t / t_p \right)^2} \leq F_{ys}$$

F_{ys}	=	Specified minimum yield strength of the stiffener (ksi)
b_t	=	Projecting width of the stiffener (in.)

t_p = Thickness of the projecting stiffener element (in.)

If the shear force in one of both panels is such that the web post-buckling or tension-field resistance is required, the moment of inertia of the transverse stiffener need only satisfy the second equation presented above.

For single-sided stiffeners, a significant portion of the web is implicitly assumed to contribute to the bending rigidity so that the neutral axis of the stiffener is assumed to be located close to the edge in contact with the web. Therefore, for this case, the moment of inertia is taken about this edge and the contribution of the web to the moment of inertia about the neutral axis is neglected for simplicity.

Transverse stiffeners used in panels with longitudinal web stiffeners must also satisfy the following relationship:

$$I_t \geq \left(\frac{b_t}{b_l}\right)\left(\frac{D}{3d_o}\right)I_l$$

Where:

I_t = Moment of inertia of the transverse web stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (in.^{4})

b_t = Projecting width of the transverse stiffener (in.)

b_λ = Projecting width of the longitudinal stiffener (in.)

D = Web depth (in.)

d_o = Smaller of the adjacent web panel widths (in.)

I_λ = Moment of inertia of the longitudinal stiffener determined as specified in **LRFD [6.10.11.3.3]** (in.^{4})

Additional information and equations used for LRFD design of transverse intermediate stiffeners are presented in **LRFD [6.10.11.1]**. In addition, a design example for transverse intermediate stiffeners is also provided in this *Bridge Manual*.

24.11 Longitudinal Stiffeners

The design of longitudinal web stiffeners is specified in **LRFD [6.10.11.3]**. Longitudinal stiffeners are aligned horizontally on the web along the length of the girder and divide the web panel into smaller sub-panels. As specified in **LRFD [6.10.2.1]**, longitudinal stiffeners are required whenever the web slenderness D/t_w exceeds 150. They are used to provide additional bend-buckling resistance to the webs of deeper girders. Longitudinal stiffeners, where required, are to consist of a plate welded to one side of the web or a bolted angle.

As specified in **LRFD [6.10.11.3.1]**, longitudinal stiffeners are to be located vertically on the web such that adequate web bend-buckling resistance is provided for constructibility and at the service limit state. It also must be verified that the section has adequate nominal flexural resistance at the strength limit state with the longitudinal stiffener in the selected position.

At composite sections in negative flexure and non-composite sections, it is recommended that the longitudinal stiffener initially be located at $0.4D_c$ from the inner surface of the compression flange. For composite sections in negative flexure, D_c would be conservatively calculated for the section consisting of the steel girder plus the longitudinal reinforcement. For non-composite sections, D_c would be based on the section consisting of the steel girder alone. As a preliminary approximation, a distance of 1/5 of the depth of the web may be used as the distance from the longitudinal stiffener to the inner surface of the compression flange.

On the exterior members, the longitudinal stiffeners are placed on the outside face of the web as shown on Standard for Plate Girder Details. If the longitudinal stiffener is required throughout the length of span on an interior member, the longitudinal stiffener is placed on one side of the web and the transverse stiffeners on the opposite side of the web. Longitudinal stiffeners are normally used in the haunch area of long spans and on a selected basis in the uniform depth section.

Where longitudinal stiffeners are used, place intermediate transverse stiffeners next to the web splice plates at a field splice. The purpose of these stiffeners is to prevent web buckling before the girders are erected and spliced.

In some cases, particularly in regions of stress reversal, it may be necessary or desirable to use two longitudinal stiffeners on the web. It is possible to have an overlap of longitudinal stiffeners near the top flange and near the bottom flange due to the variation between maximum positive and maximum negative moment.

It is preferred that longitudinal stiffeners be placed on the opposite side of the web from transverse stiffeners. At bearing stiffeners and connection plates where the longitudinal stiffener and transverse web element must intersect, a decision must be made as to which element to interrupt. According to **LRFD [6.10.11.3.1]**, wherever practical, longitudinal stiffeners are to extend uninterrupted over their specified length, unless otherwise permitted in the contract documents, since longitudinal stiffeners are designed as continuous members to improve the web bend buckling resistance. In such cases, the interrupted transverse elements must be fitted and attached to both sides of the longitudinal stiffener with connections sufficient to develop the flexural and axial resistance of the transverse element. If the longitudinal stiffener is interrupted instead, it should be similarly attached to all transverse elements. All interruptions must be carefully designed with respect to fatigue, especially if the longitudinal stiffener is not attached to the transverse web elements, as a Category E or E′ detail may exist at the termination points of each longitudinal stiffener-to-web weld. Copes should always be provided to avoid intersecting welds.

Longitudinal stiffeners are subject to the same flexural strain as the web at their vertical position on the web. As a result, the stiffeners must have sufficient strength and rigidity to resist bend buckling of the web (at the appropriate limit state) and to transmit the stresses in the stiffener and an effective portion of the web as an equivalent column. Therefore, as specified in **LRFD [6.10.11.3.1]**, the flexural stress in the longitudinal stiffener due to the factored loads, f_s, must satisfy the following at the strength limit state and when checking constructability:

$$f_s \leq \phi_f R_h F_{ys}$$

Where:

ϕ_f	=	Resistance factor for flexure specified in **LRFD [6.5.4.2]** (= 1.0)
R_h	=	Hybrid factor specified in **LRFD [6.10.1.10.1]**
F_{ys}	=	Specified minimum yield strength of the longitudinal stiffener (ksi)

24.11.1 Projecting Width

As specified in **LRFD [6.10.11.3.2]**, the projecting width, b_λ, of the longitudinal stiffener must satisfy the following requirement in order to prevent local buckling of the stiffener plate:

$$b_l \leq 0.48 t_s \sqrt{\frac{E}{F_{ys}}}$$

Where:

t_s	=	Thickness of the longitudinal stiffener (in.)
F_{ys}	=	Specified minimum yield strength of the stiffener (ksi)

24.11.2 Moment of Inertia

As specified in **LRFD [6.10.11.3.3]**, to ensure that a longitudinal stiffener will have adequate rigidity to maintain a horizontal line of near zero lateral deflection in the web to resist bend buckling of the web (at the appropriate limit state), the moment of inertia of the stiffener acting in combination with an adjacent strip of web must satisfy the following requirement:

$$I_l \geq D t_w^3 \left[2.4 \left(\frac{d_o}{D} \right)^2 - 0.13 \right] \beta$$

Where:

I_λ = Moment of inertia of the longitudinal stiffener including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (in.^{4}). If F_{yw} is smaller than F_{ys}, the strip of web included in the effective section must be reduced by the ratio of F_{yw}/F_{ys}.

D = Web depth (in.)

t_w = Web thickness (in.)

d_o = Transverse stiffener spacing (in.)

β = Curvature correction factor for longitudinal stiffener rigidity (equal to 1.0 for longitudinal stiffeners on straight webs)

Longitudinal stiffeners on horizontally curved webs require greater rigidity than on straight webs because of the tendency of curved webs to bow. This is reflected by including the factor β in the above equation, which is a simplification of a requirement for longitudinal stiffeners on curved webs. For longitudinal stiffeners on straight webs, β equals 1.0.

The moment of inertia (and radius of gyration) of the longitudinal stiffener is taken about the neutral axis of an equivalent column cross section consisting of the stiffener and an adjacent strip of web with a width of $18t_w$.

24.11.3 Radius of Gyration

As specified in **LRFD [6.10.11.3.3]**, to ensure that the longitudinal stiffener acting in combination with an adjacent strip of web as an effective column section can withstand the axial compressive stress without lateral buckling, the radius of gyration, r, of the effective column section must satisfy the following requirement:

$$r \geq \frac{0.16 d_o \sqrt{\frac{F_{ys}}{E}}}{\sqrt{1 - 0.6 \frac{F_{yc}}{R_h F_{ys}}}}$$

Where:

r = Radius of gyration of the longitudinal stiffener including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (in.)

d_o = Transverse stiffener spacing (in.)

F_{ys} = Specified minimum yield strength of the longitudinal stiffener (ksi)

F_{yc} = Specified minimum yield strength of the compression flange (ksi)

R_h = Hybrid factor determined as specified in **LRFD [6.10.1.10.1]**

Additional information and equations used for LRFD design of longitudinal stiffeners are presented in **LRFD [6.10.11.3]**.

24.12 Construction

When the deck slab is poured, the exterior girder tends to rotate between the diaphragms. This problem may result if the slab overhang is greater than recommended and/or if the girders are relatively shallow in depth. This rotation causes the rail supporting the finishing machine to deflect downward and changes the roadway grade unless the contractor provides adequate lateral timber bracing.

Stay-in-place steel forms are not recommended for use. Steel forms have collected water that permeates through the slab and discharges across the top flanges of the girders. As a result, flanges frequently corrode. Since there are cracks in the slab, this is a continuous problem.

Where built-up box sections are used, full penetration welds provide a stronger joint than fillet welds and give a more aesthetically pleasing appearance. However, they are also more costly.

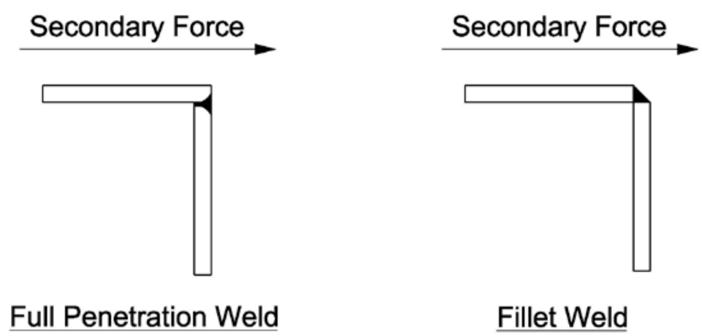

Figure 24.12-1
Welds for Built-up Box Sections

The primary force of the member is tension or compression along the axis of the member. The secondary force is a torsional force on the member cross section which produces a shearing force across the weld.

During construction, holes may be drilled in the top flanges in the compression zone to facilitate anchorage of posts for safety lines. The maximum hole size is 3/4" diameter, and prior to pouring the concrete deck, a bolt must be placed in each hole.

LRFD [6.10.3] describes the constructability design requirements for a steel girder bridge. Provisions are provided for the following constructability checks:

- Nominal yielding
- Reliance on post-buckling resistance
- Potential uplift at bearings
- Webs without bearings stiffeners
- Holes in tension flanges

- Load-resisting bolted connections
- Flexure in discretely braced flanges
- Flexure in continuously braced flanges
- Shear in interior panels of webs with transverse stiffeners
- Dead load deflections

24.12.1 Web Buckling

The buckling behavior of a slender web plate subject to pure bending is similar to the buckling behavior of a flat plate. Through experimental tests, it has been observed that web bend-buckling behavior is essentially a load-deflection rather than a bifurcation phenomenon; that is, a distinct buckling load is not observed.

Since web plates in bending do not collapse when the theoretical buckling load is reached, the available post-buckling strength can be considered in determining the nominal flexural resistance of sections with slender webs at the strength limit state. However, during the construction condition, it is desirable to limit the bending deformations and transverse displacements of the web.

The advent of composite design has led to a significant reduction in the size of compression flanges in regions of positive flexure. As a result, more than half of the web of the non-composite section will be in compression in these regions during the construction condition before the concrete deck has hardened or is made composite. As a result, the web is more susceptible to bend-buckling in this condition.

To control the web plate bending strains and transverse displacements during construction, *AASHTO LRFD* uses the theoretical web bend-buckling load as a simple index. The web bend-buckling resistance, F_{crw}, is specified in **LRFD [6.10.1.9.1]** as follows:

$$Fcrw = \frac{0.9Ek}{\left(D/t_w \right)^2}$$

Where:

E	=	Modulus of elasticity of the steel (ksi)
K	=	Bend-buckling coefficient (see below)
D	=	Depth of web (in.)
t_w	=	Thickness of web (in.)

For webs without longitudinal stiffeners, the bend-buckling coefficient, k, is as follows:

$$k = \frac{9}{\left(D_c / D\right)^2}$$

Where:

D_c = Depth of web in compression in the elastic range (in.)

F_{crw} is not to exceed the smaller of R_hF_{yc} and $F_{yw}/0.7$, where F_{yc} and F_{yw} are the specified minimum yield strengths of the compression flange and web, respectively, and R_h is the hybrid factor.

According to **LRFD [6.10.3.2]**, the maximum compression-flange stress in a non-composite I-section due to the factored loads, calculated without consideration of flange lateral bending, must not exceed the resistance factor for flexure, ϕ_f, times F_{crw} for all critical stages of construction. This requirement also applies at sections where top flanges of tub girders are subject to compression during construction. For closed-box sections, **LRFD [6.11.3.2]** states that the maximum longitudinal flange stress due to the factored loads, calculated without consideration of longitudinal warping, must not exceed ϕ_fF_{crw} at sections where non-composite box flanges are subject to compression during construction. (A box flange is defined in *AASHTO LRFD* as a flange connected to two webs.) For tub or closed-box sections with inclined webs, D_c should be taken as the depth of the web in compression measured along the slope (that is, D_c divided by the cosine of the angle of inclination of the web plate with respect to the vertical) when computing F_{crw}. Should F_{crw} be exceeded for the construction condition, the engineer has several options to consider:

- Provide a larger compression flange or a smaller tension flange to reduce D_c.
- Adjust the deck-placement sequence to reduce the compressive stress in the web.
- Provide a thicker web.
- As a last resort, should the previous options not prove practical or cost-effective, provide a longitudinal web stiffener.

24.12.2 Deck Placement Analysis

Depending on the length of the bridge, the construction of the deck may require placement in sequential stages. Therefore, certain sections of the steel girders will become composite before other sections. If certain placement sequences are followed, temporary moments induced in the girders during the deck placement can be significantly higher than the final non-composite dead load moments after the sequential placement is complete.

Therefore, **LRFD [6.10.3.4]** requires that sections in positive flexure that are non-composite during construction but composite in the final condition must be investigated for flexure according to the provisions of **LRFD [6.10.3.2]** during the various stages of the deck placement. Furthermore, changes in the load, stiffness and bracing during the various stages are to be considered in the analysis.

Example:

Consider the sample deck placement shown in Figure 24.12-2 for a three-span continuous bridge. The deck placement sequence is based on Standard for Slab Pouring Sequence.

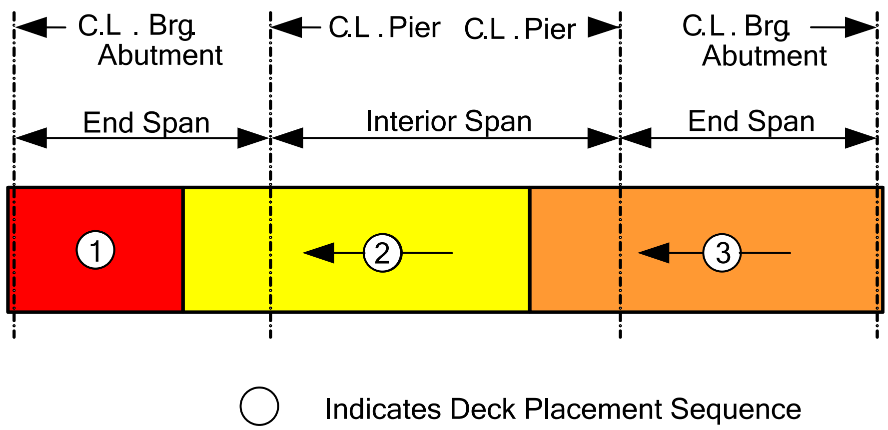

Figure 24.12-2
Deck Placement Sequence

Figure 24.12-3 through Figure 24.12-6 show elevation views of a girder which will be used to show the results for each stage of the deck placement sequence assumed for this example in Figure 24.12-2. In Figure 24.12-3, the girders are in place but no deck concrete has yet been placed. The entire girder length is non-composite at this stage. Before the deck is placed, the non-composite girder must resist the moments due to the girder self-weight and any additional miscellaneous steel weight. The moments due to these effects are shown at Location A, which is the location of maximum positive moment in the first end span.

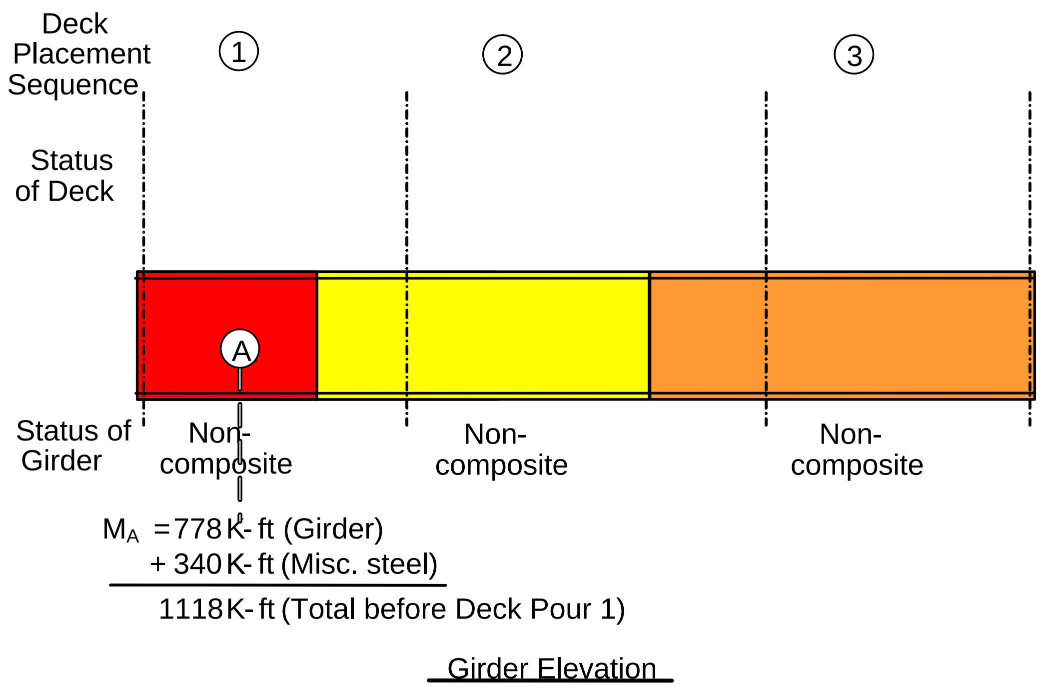

Figure 24.12-3
Girder Elevation View

Figure 24.12-4 shows the first deck placement (Cast 1), which is cast in the first portion of the first span. The moment due to the wet concrete load, which consists of the weight of the deck and deck haunches, is added to the moments due to the girder self-weight and miscellaneous steel weight. Since the concrete in this first placement has not yet hardened, the moment due to the first deck placement is resisted by the non-composite girder. The cumulative positive moment in the girder at Location A after the first deck placement is +3,565 kip-ft, which is the maximum positive moment this section will experience during the assumed placement sequence. This moment is larger than the moment of +3,542 kip-ft that would be computed at this location assuming a simultaneous placement of the entire deck (that is, ignoring the sequential stages).

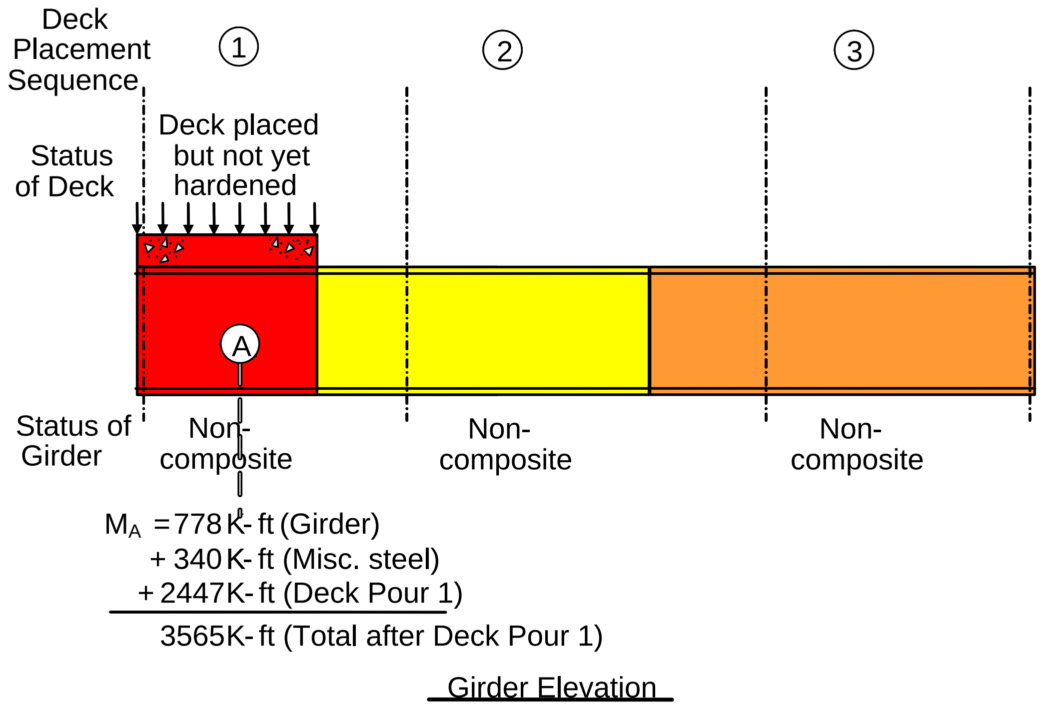

Figure 24.12-4
Deck Placement Analysis 1

The next deck placement (Cast 2) is located immediately adjacent to Cast 1, as shown in Figure 24.12-5. The concrete in the first placement is now assumed to be hardened so that those portions of the girder are now composite. Therefore, as required in **LRFD [6.10.3.4]**, those portions of the girder are assumed composite in the analysis for this particular deck placement. The remainder of the girder is non-composite. Since the deck casts are relatively short-term loadings, the short-term modular ratio, n, is used to compute the composite stiffness. The previous casts are assumed to be fully hardened in this case, but adjustments to the composite stiffness to reflect the actual strength of the concrete in the previous casts at the time of this particular placement could be made, if desired. The cumulative moment at Location A has decreased from +3,565 kip-ft after Cast 1 to +3,449 kip-ft after Cast 2, because the placement in Cast 2 causes a negative moment in the end spans.

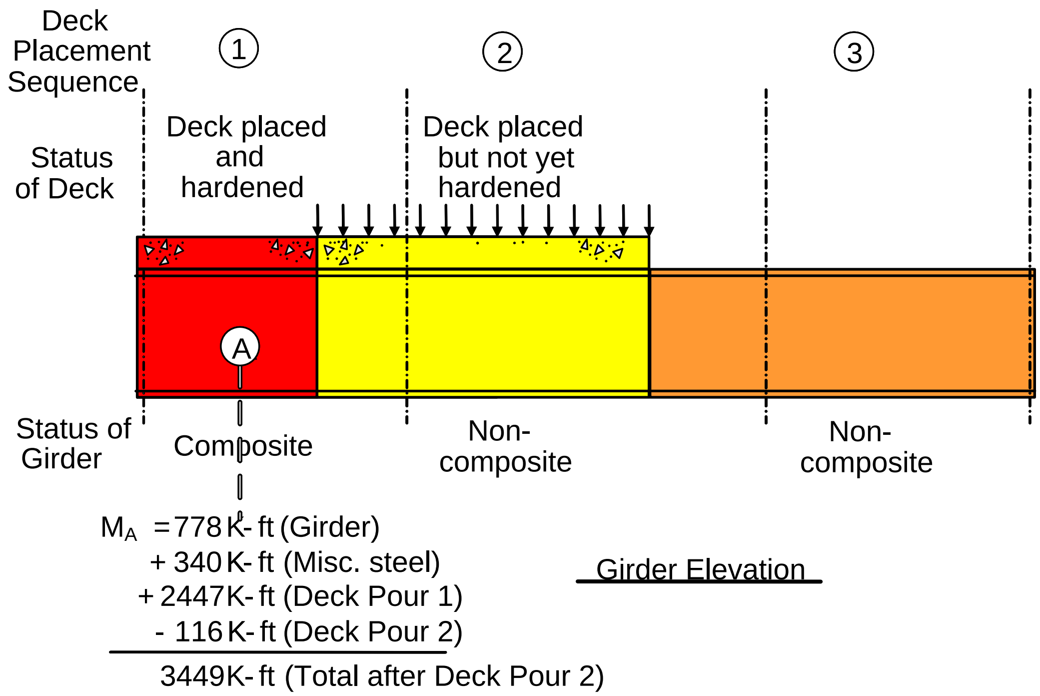

Figure 24.12-5
Deck Placement Analysis 2

The last deck placement (Cast 3) is located immediately adjacent to Cast 2, as presented in Figure 24.12-6. Again, the concrete in Casts 1 and 2 is assumed to be fully hardened in the analysis for Cast 3. The cumulative moment at Location A has increased slightly from +3,449 kip-ft to +3,551 kip-ft, which is less than the moment of +3,565 kip-ft experienced at Location A after Cast 1.

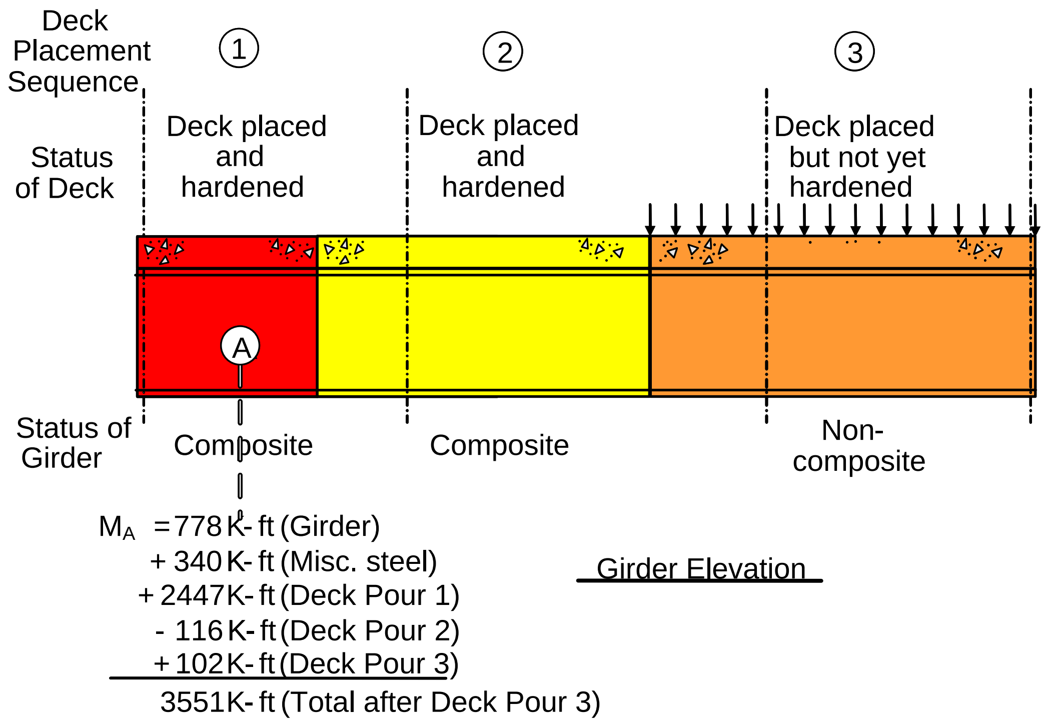

Figure 24.12-6
Deck Placement Analysis 3

Table 24.12-1 shows a more complete set of the unfactored dead-load moments in the end span (Span 1) from the abutment to the end of Cast 1 computed from the example deck placement analysis. Data are given at 19.0-foot increments along the span, measured from the abutment. The end of Cast 1 is located 102.5 feet from the abutment, based on the requirements of Standard for Slab Pouring Sequence. Location A is 76.0 feet from the abutment. In addition to the moments due to each of the individual casts, Table 24.12-1 gives the moments due to the steel weight and the additional miscellaneous steel. Also included are the sum of the moments due to the three casts and the moments due to the weight of the concrete deck and haunches assuming that the concrete is placed simultaneously on the non-composite girders instead of in sequential steps. The maximum moment occurs after Cast 1.

Length (ft)	0.0	19.0	38.0	57.0	76.0	95.0
Steel Weight	0	400	663	789	778	630
Additional Miscellaneous Steel	0	166	278	336	340	290
Cast 1	0	1190	1994	2413	2447	2096
Cast 2	0	-29	-58	-87	-116	-145
Cast 3	0	25	51	76	102	127
Sum of Casts	0	1186	1987	2402	2433	2078
Deck & Haunches (Simultaneous Cast)	0	1184	1983	2396	2424	2067

Table 24.12-1

Moments from Deck Placement Analysis (K-ft)

The slight differences in the moments on the last line of Table 24.12-1 (assuming a simultaneous placement of the entire slab) and the sum of the moments due to the three casts are due to the changes in the girder stiffness with each sequential cast. The principle of superposition does not apply directly in the deck-placement analyses, since the girder stiffness changes at each step of the analysis. Although the differences in the moments are small in this example, they can be significantly greater depending on the span configuration. The effects of the deck placement sequence must be considered during design.

In regions of positive flexure, the non-composite girder should be checked for the effect of the maximum accumulated deck-placement moment. This moment at 76 feet from Abutment 1 is computed as:

$$M = 778 + 340 + 2{,}447 = 3{,}565 \text{ kip-ft}$$

This value agrees with the moment at this location shown in Figure 24.12-4.

In addition to the dead load moments during the deck placement, unfactored dead load deflections and reactions can also be investigated similarly during the construction condition.

When investigating reactions during the construction condition, if uplift is found to be present during deck placement, the following options can be considered:

- Rearrange the concrete casts.
- Specify a temporary load over that support.
- Specify a tie-down bearing.

- Perform another staging analysis with zero bearing stiffness at the support experiencing lift-off.

24.13 Painting

The final coat of paint on all steel bridges shall be an approved color. Exceptions to this policy may be considered on an individual basis for situations such as scenic river crossings, unique or unusual settings or local community preference. The Region is to submit requests for an exception along with the Structure Survey Report. The colors available for use on steel structures are shown in Chapter 9 - Materials.

Unpainted weathering steel is used on bridges over streams and railroads. All highway grade separation structures require the use of painted steel, since unpainted steel is subject to excessive weathering from salt spray distributed by traffic. On weathering steel bridges, the end 6' of any steel adjacent to either side of an expansion joint and/or hinge is required to have two shop coats of paint. The second coat is to be brown color similar to rusted steel. Do not paint the exterior face of the exterior girders for aesthetic reasons, but paint the hanger bar on the side next to the web. Additional information on painting is presented in Chapter 9 - Materials.

For painted steel plate I-girders utilize a three-coat system defined by the Standard Specification bid item "Painting Epoxy System (Structure)". For painted tub girders use a two-coat system defined by the STSP "Painting Polysiloxane System (Structure)", which includes painting of the inside of the tubs.

Paint on bridges affects the slip resistance of bolted connections. Since faying surfaces that are not galvanized are typically blast-cleaned as a minimum, a Class A surface condition should only be used to compute the slip resistance when Class A coatings are applied or when unpainted mill scale is left on the faying surface. Most commercially available primers will qualify as Class B coatings.

24.14 Floor Systems

In the past, floor systems utilizing two main girders were used on long span structures. Current policy is to use multiple plate girder systems for bridges having span lengths up to 400'. Multiple girder systems are preferred since they are redundant; that is, failure of any single member will not cause failure of the structure.

In a two-girder system, the main girders are designed equally to take the dead load and live load unless the roadway cross section is unsymmetrical. The dead load and live load carried by the intermediate stringers is transferred to the floor beams, which transmit the load to the main girders. In designing the main girders, it is an acceptable practice to assume the same load distribution along the stringers as along the girder and ignore the concentrated loads at the floor beam connections.

The design criteria used for such girders is the same as the criteria used for plate girders and rolled sections. Particular attention should be paid to the sufficiency of the girder connection details and to the lateral bracing requirements and connections.

24.15 Box Girders

Box girders present a smooth, uncluttered appearance under the bridge deck due to the lack of transverse bracing and due to their closed section. Enhanced torsional rigidity can make box girders a favorable choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

In the design of box girders, the concrete slab is designed as a portion of the top flange and also as the support between the two girder webs which satisfies the requirement for being considered a closed box section.

During construction, the box girder requires additional bracing to support the web until the slab is in place. The additional bracing can either be temporary bracing that is removed after the slab is cured or permanent bracing that is left in place. In either case, it is an additional cost item. In addition, temporary bracing is required between the girders to transfer wind loads until the deck slab is poured.

Current experience shows that box girders may require more material than conventional plate girders. On longer spans, additional bracing between girders is required to transfer lateral loads.

Several requirements in *AASHTO LRFD* are specific to box girders. For box girders, sections in positive flexure shall not have a yield strength in excess of 70 ksi. The following web slenderness requirement from **LRFD [6.11.6.2.2]** must also be satisfied:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$

Where:

D_{cp} = Depth of web in compression at plastic moment (in.)

t_w = Web thickness (in.)

F_{yc} = Specified minimum yield strength of the compression flange (ksi)

Other requirements for positive flexure in box girders are presented in **LRFD [6.11.6.2.2]**. Steel sections in negative flexure must not use the provisions in Appendices A or B of the *AASHTO LRFD* specifications.

When computing effective flange widths for closed-box sections, the distance between the outside of the webs at the tops is to be used in lieu of the web thickness in the general requirements. For closed box sections, the spacing should be taken as the spacing between the centerlines of the box sections.

When computing section properties for closed-box sections with inclined webs, the moment of inertia of the webs about a horizontal axis at the mid-depth of the web should be adjusted for the web slope by dividing by the cosine of the angle of inclination of the web plate to the vertical. Also, inspection manholes are often inserted in the bottom flanges of closed-box sections near supports. These manholes should be subtracted from the bottom-flange area when computing the elastic section properties for use in the region of the access hole. If longitudinal flange stiffeners are present on the closed-box section, they are often included when computing the elastic section properties.

When investigating web bend-buckling resistance for closed-box sections, **LRFD [6.11.3.2]** states that the maximum longitudinal flange stress due to the factored loads, calculated without consideration of longitudinal warping, must not exceed $\phi_f F_{crw}$ at sections where non-composite box flanges are subject to compression during construction. For more information about the web bend-buckling resistance of box girders, refer to 24.12.1. In *AASHTO LRFD*, a box flange is defined as a flange connected to two webs.

Torsion in structural members is generally resisted through a combination of St. Venant torsion and warping torsion. For closed cross-sections such as box girders, St. Venant torsion generally dominates. Box girders possess favorable torsional characteristics which make them an attractive choice for horizontally curved bridges. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

WisDOT policy item:

Rigorous analysis of single-box and two-box girder bridges to eliminate the need for in-depth fracture critical inspections is not allowed.

Full-depth, solid diaphragms between box girders shall be used at the one-third points (min.) in each span.

BOS approval is required for all box girders.

24.16 Design Examples

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Table of Contents

E24-1 2-Span Continous Steel Plate Girder Bridge - LRFD

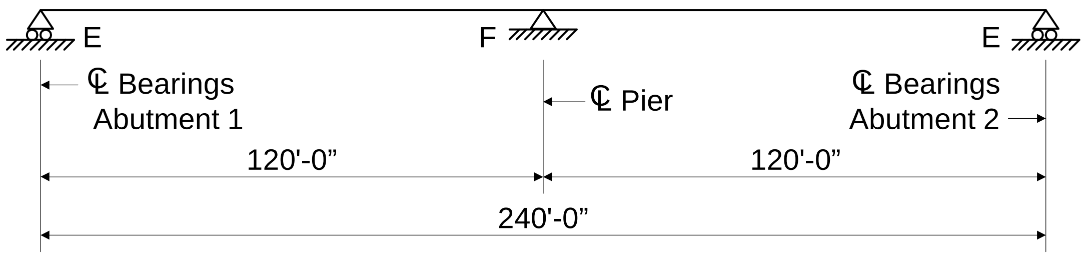
This example shows design calculations conforming to the *AASHTO LRFD Seventh Edition -2014* as supplemented by the *WisDOT Bridge Manual* . Sample design calculations are shown for the following steel superstructure regions or components:

- Interior girder design at the controlling positive moment region
- Interior girder design at the controlling negative moment region
- Transverse stiffener design
- Shear connector design
- Bearing stiffener design

E24-1.1 Obtain Design Criteria

The first design step for a steel girder is to choose the correct design criteria. [24.6.1]

The steel girder design criteria are obtained from Figure E24-1.1-1 through Figure E24-1.1-3 (shown below), and from the referenced articles and tables in the *AASHTO LRFD Bridge Design Specifications, Seventh Edition*. An interior plate girder will be designed for an HL-93 live load for this example. The girder will be designed to be composite throughout. (Note: Figure 5.2-1 contains recommended economical span lengths for steel girders.)



Legend:
E = Expansion Bearings
F = Fixed Bearings

Figure E24-1.1-1
Span Configuration

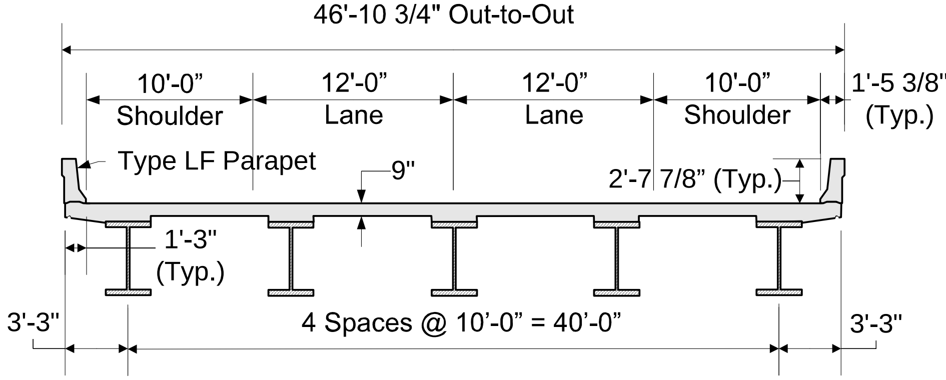


Figure E24-1.1-2
Superstructure Cross Section

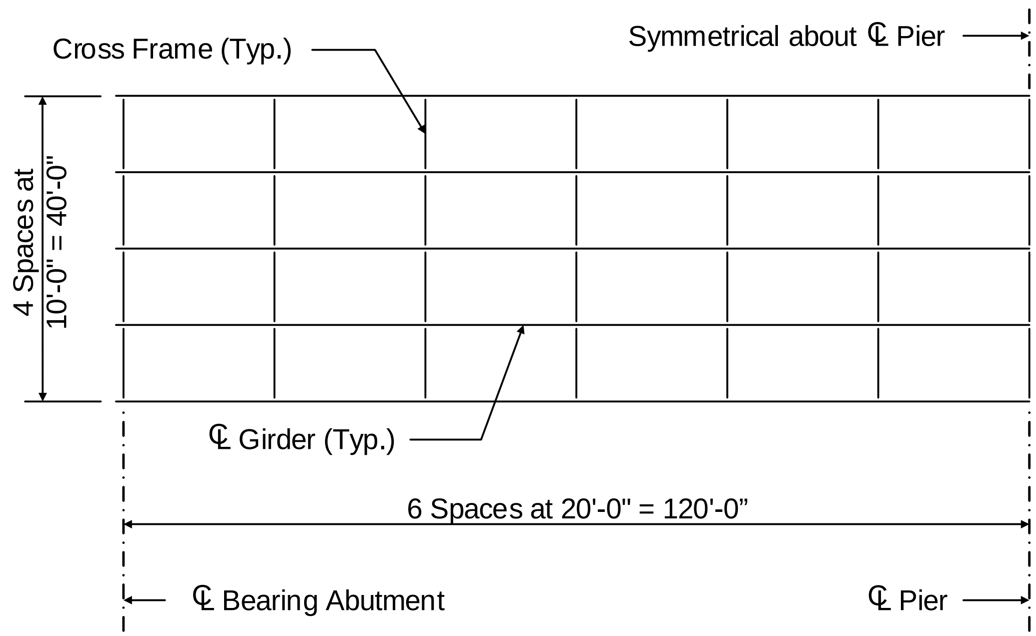


Figure E24-1.1-3
Framing Plan

Design criteria:

$N_{spans} := 2$	Number of spans	
$L := 120$	ft	span length
$Skew := 0$	deg	skew angle
$N_b := 5$	number of girders	
$S := 10.0$	ft	girder spacing
$S_{overhang} := 3.25$	ft	deck overhang (Per Chapter 17.6.2, WisDOT practice is to limit the overhang to 3'-7", however, economical overhang range is 0.28S - 0.35S based on parapet weight.)
$L_b := 240$	in	cross-frame spacing **LRFD [6.7.4]**
$F_{yw} := 50$	ksi	web yield strength **LRFD [Table 6.4.1-1]**
$F_{yf} := 50$	ksi	flange yield strength **LRFD [Table 6.4.1-1]**
$f'_c := 4.0$	ksi	concrete 28-day compressive strength **LRFD [5.4.2.1 & Table C5.4.2.1-1]**
$f_y := 60$	ksi	reinforcement strength **LRFD [5.4.3 & 6.10.1.7]**

$E_s := 29000$	ksi	modulus of elasticity **LRFD [6.4.1]**
$t_{deck} := 9.0$	in	total deck thickness
$t_s := 8.5$	in	effective deck thickness
$t_{overhang} := 9.5$	in	total overhang thickness
$t_{effoverhang} := 9.0$	in	effective overhang thickness
$W_s := 0.490$	kcf	steel density **LRFD [Table 3.5.1-1]**
$w_c := 0.150$	kcf	concrete density **LRFD [Table 3.5.1-1 & C3.5.1]**
$DL_{misc} := 0.030$	kip/ft	additional miscellaneous dead load (per girder) (Chapter 17.2.4.1)
$W_{par} := 0.464$	kip/ft	parapet weight (each) (Type 32SS)
$W_{fws} := 0.020$	ksf	future wearing surface (Chapter 17.2.4.1)
$w_{deck} := 46.50$	ft	deck width
$w_{roadway} := 44.0$	ft	roadway width
$d_{haunch} := 3.75$	in	haunch depth (from top of web for design) (for construction, the haunch is measured from the top of the top flange)
$ADTT_{SL} := 3000$	Average Daily Truck Traffic (Single-Lane)	

Design factors from AASHTO LRFD Bridge Design Specifications:

Load factors, γ, **LRFD [Table 3.4.1-1 & Table 3.4.1-2]**:

Load Combinations and Load Factors							
Limit State	Load Factors						
	DC	DW	LL	IM	WS	WL	EQ
Strength I	1.25	1.50	1.75	1.75	-	-	-
Service II	1.00	1.00	1.30	1.30	-	-	-
Fatigue I	-	-	1.50	1.50	-	-	-

Table E24-1.1-1
Load Combinations and Load Factors

The abbreviations used in Table E24-1.1-1 are as defined in **LRFD [3.3.2]**.

The extreme event limit state (including earthquake load) is generally not considered for a steel girder design.

Resistance factors,ϕ, **LRFD [6.5.4.2]**:

Resistance Factors	
Type of Resistance	**Resistance Factor**
For flexure	1.00
For shear	1.00
For axial compression	0.90

Table E24-1.1-2
Resistance Factors

Dynamic load allowance **LRFD [Table 3.6.2.1-1]**:

Dynamic Load Allowance	
Limit State	**Dynamic Load Allowance, IM**
Fatigue and Fracture Limit State	15%
All Other Limit States	33%

Table E24-1.1-3
Dynamic Load Allowance

E24-1.2 Select Trial Girder Section

Before the dead load effects can be computed, a trial girder section must be selected. [24.6.2] This trial girder section is selected based on previous experience and based on preliminary design. For this design example, the trial girder section presented in Figure E24-1.2-1 will be used. Based on this trial girder section, section properties and dead load effects will be computed. Then specification checks will be performed to determine if the trial girder section successfully resists the applied loads. If the trial girder section does not pass all specification checks or if the girder optimization is not acceptable, then a new trial girder section must be selected and the design process must be repeated.

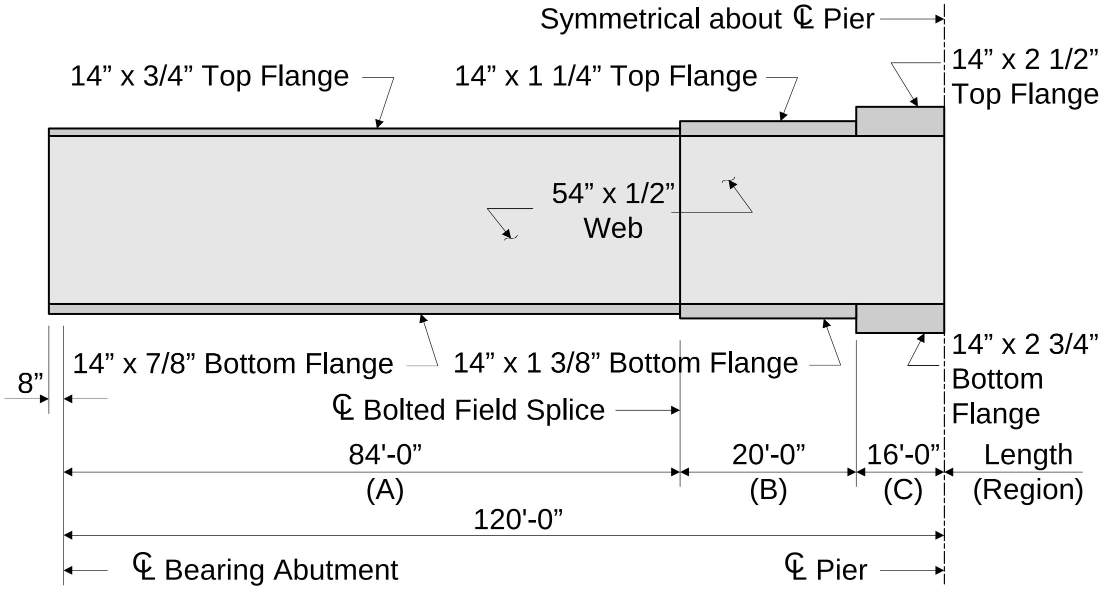


Figure E24-1.2-1
Plate Girder Elevation

The AASHTO/NSBA Steel Bridge Collaboration Document "Guidelines for Design for Constructibility" recommends a 3/4" minimum flange thickness. Wisconsin requires a 3/4" minimum flange thickness.

E24-1.3 Compute Section Properties

Since the superstructure is composite, several sets of section properties must be computed **LRFD [6.10.1.1]**. The initial dead loads (or the noncomposite dead loads) are applied to the girder-only section. For permanent loads assumed to be applied to the long-term composite section, the long-term modular ratio of 3n is used to transform the concrete deck area **LRFD [6.10.1.1.1b]**. For transient loads assumed applied to the short-term composite section, the short-term modular ratio of n is used to transform the concrete deck area.

For girders with shear connectors provided throughout their entire length and with slab reinforcement satisfying the provisions of **LRFD [6.10.1.7]**, stresses due to loads applied to the composite section for the fatigue limit state may be computed using the short-term composite section assuming the concrete slab to be fully effective for both positive and negative flexure **LRFD [6.6.1.2.1 & 6.10.5.1]**.

For girders with shear connectors provided throughout their entire length that also satisfy the provisions of **LRFD [6.10.1.7]**, flexural stresses caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete deck is effective for both positive and negative flexure **LRFD [6.10.4.2.1]**.

In general, both the exterior and interior girders must be considered, and the controlling design is used for all girders, both interior and exterior. However, design computations for the interior girder only are presented in this example.

The modular ratio, n, is computed as follows:

$$n := \frac{E_s}{E_c}$$

Where:

E_s = Modulus of elasticity of steel (ksi)

E_c = Modulus of elasticity of concrete (ksi)

$E_s = 29000$ ksi **LRFD [6.4.1]**

$$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$$ **LRFD [5.4.2.4]**

Where:

K_1 = Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction. For WisDOT, K_1 = 1.0.

w_c = Unit weight of concrete (kcf)

f'_c = Specified compressive strength of concrete (ksi)

$w_c = 0.150$ kcf **LRFD [Table 3.5.1-1 & C3.5.1]**

$f'_c = 4.0$ ksi **LRFD [Table 5.4.2.1-1 & 5.4.2.1]**

$K_1 := 1$ **LRFD [5.4.2.4]**

$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ $\boxed{E_c = 3834}$ ksi

$n := \frac{E_s}{E_c}$ $\boxed{n = 7.6}$ **LRFD [6.10.1.1.1b]**

Therefore, use: $n := 8$

The effective flange width is computed as follows (Chapter 17.2.11):

For interior beams, the effective flange width is taken as the average spacing of adjacent beams:

$W_{effflange} := S$ $\boxed{W_{effflange} = 10.00}$ ft

or

$\boxed{W_{effflange} \cdot 12 = 120.00}$ in

Based on Table 17.5-3 of Chapter 17 for a 9" deck and 10'-0" girder spacing, the top mat longitudinal continuity reinforcement bar size and spacing is #6 bars at 7.5" spacing. The area of the top mat longitudinal continuity deck reinforcing steel in the negative moment region is computed below for the effective flange width. For the section properties in Table E24-1.3-3, the location of the centroid of the top mat longitudinal reinforcement is conservatively taken as one-half the structural slab thickness or 8.5" / 2 = 4.25".

$$A_{deckreinf} := 1 \times 0.44 \cdot \frac{W_{effflange} \cdot 12}{7.5} \qquad \boxed{A_{deckreinf} = 7.04} \quad in^2$$

For this design example, the slab haunch is 3.75 inches throughout the length of the bridge. That is, the bottom of the slab is located 3.75 inches above the top of the web (for construction, it is measured from the top of the top flange). For this design example, this distance is used in computing the location of the centroid of the slab. However, the area of the haunch is conservatively not considered in the section properties for this example.

Based on the trial plate sizes shown in Figure E24-1.2-1, the noncomposite and composite section properties for Region A, B, and C are computed as shown in the following tables **LRFD [6.6.1.2.1, 6.10.5.1, 6.10.4.2.1]**. The distance to the centroid is measured from the bottom of the girder.

Region A Section Properties (0 - 84 Feet)						
Section	**Area, A (inches^2)**	**Centroid, d (inches)**	**A*d (inches^3)**	**I_o (inches^4)**	**A*y^2 (inches^4)**	**I_{total} (inches^4)**
Girder only:						
Top flange	10.500	55.250	580.1	0.5	8441.1	8441.6
Web	27.000	27.875	752.6	6561.0	25.8	6586.8
Bottom flange	12.250	0.438	5.4	0.8	8576.1	8576.9
Total	49.750	26.897	1338.1	6562.3	17043.0	23605.3
Composite (3n):						
Girder	49.750	26.897	1338.1	23605.3	13668.5	37273.7
Slab	42.500	62.875	2672.2	255.9	16000.2	16256.0
Total	92.250	43.472	4010.3	23861.1	29668.6	53529.8
Composite (n):						
Girder	49.750	26.897	1338.1	23605.3	33321.4	56926.6
Slab	127.500	62.875	8016.6	767.7	13001.9	13769.5
Total	177.250	52.777	9354.7	24372.9	46323.2	70696.2
Section	**y_{botgdr} (inches)**	**y_{topgdr} (inches)**	**$y_{topslab}$ (inches)**	**S_{botgdr} (inches^3)**	**S_{topgdr} (inches^3)**	**$S_{topslab}$ (inches^3)**
Girder only	26.897	28.728	---	877.6	821.7	---
Composite (3n)	43.472	12.153	23.653	1231.4	4404.7	2263.1
Composite (n)	52.777	2.848	14.348	1339.5	24820.6	4927.1

Table E24-1.3-1
Region A Section Properties

Region B Section Properties (84 - 104 Feet)						
Section	**Area, A (inches^2)**	**Centroid, d (inches)**	**A*d (inches^3)**	**I_o (inches^4)**	**A*y^2 (inches^4)**	**I_{total} (inches^4)**
Girder only:						
Top flange	17.500	56.000	980.0	2.3	14117.0	14119.3
Web	27.000	28.375	766.1	6561.0	16.3	6577.3
Bottom flange	19.250	0.688	13.2	3.0	13940.2	13943.2
Total	63.750	27.598	1759.4	6566.3	28073.5	34639.8
Composite (3n):						
Girder	63.750	27.598	1759.4	34639.8	13056.1	47695.9
Slab	42.500	63.375	2693.4	255.9	19584.1	19840.0
Total	106.250	41.909	4452.8	34895.7	32640.2	67535.9
Composite (n):						
Girder	63.750	27.598	1759.4	34639.8	36266.9	70906.7
Slab	127.500	63.375	8080.3	767.7	18133.5	18901.1
Total	191.250	51.449	9839.7	35407.4	54400.4	89807.8
Section	**y_{botgdr} (inches)**	**y_{topgdr} (inches)**	**$y_{topslab}$ (inches)**	**S_{botgdr} (inches^3)**	**S_{topgdr} (inches^3)**	**$S_{topslab}$ (inches^3)**
Girder only	27.598	29.027	---	1255.2	1193.4	---
Composite (3n)	41.909	14.716	25.716	1611.5	4589.2	2626.2
Composite (n)	51.449	5.176	16.176	1745.6	17351.7	5552.0

Table E24-1.3-2
Region B Section Properties

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not.

As previously explained, for this design example, the concrete slab will be assumed to be fully effective for both positive and negative flexure for service and fatigue limit states.

Region C Section Properties (104 - 120 Feet)						
Section	**Area, A (inches^2)**	**Centroid, d (inches)**	**A*d (inches^3)**	**I_o (inches^4)**	**A*y^2 (inches^4)**	**I_{total} (inches^4)**
Girder only:						
Top flange	35.000	58.000	2030.0	18.2	30009.7	30027.9
Web	27.000	29.750	803.3	6561.0	28.7	6589.7
Bottom flange	38.500	1.375	52.9	24.3	28784.7	28809.0
Total	100.500	28.718	2886.2	6603.5	58823.1	65426.6
Composite (deck concrete using 3n):						
Girder	100.500	28.718	2886.2	65426.6	11525.0	76951.6
Slab	42.500	64.750	2751.9	255.9	27253.3	27509.2
Total	143.000	39.427	5638.1	65682.5	38778.3	104460.8
Composite (deck concrete using n):						
Girder	100.500	28.718	2886.2	65426.6	40802.5	106229.1
Slab	127.500	64.750	8255.6	767.7	32162.0	32929.6
Total	228.000	48.868	11141.8	66194.3	72964.4	139158.7
Composite (top longitudinal deck reinforcement only):						
Girder	100.500	28.718	2886.2	65426.6	559.2	65985.8
Deck reinf.	7.040	64.750	455.8	0.0	7982.4	7982.4
Total	107.540	31.077	3342.0	65426.6	8541.6	73968.2
Section	**y_{botgdr} (inches)**	**y_{topgdr} (inches)**	**y_{deck} (inches)**	**S_{botgdr} (inches^3)**	**S_{topgdr} (inches^3)**	**S_{deck} (inches^3)**
Girder only	28.718	30.532	---	2278.2	2142.9	---
Composite (3n)	39.427	19.823	29.573	2649.5	5269.7	3532.3
Composite (n)	48.868	10.382	20.132	2847.7	13403.3	6912.2
Composite (rebar	31.077	28.173	33.673	2380.2	2625.5	2196.7

Table E24-1.3-3
Region C Section Properties

The section properties used to compute the unfactored dead and live load moments and shears for each girder region are given in the following table in accordance with the requirements of **LRFD [6.10.1.5]**.

Girder Region (ft)	Moment of Inertia Used (in^4)		
	Beam Self Weight, Misc Dead Loads, Concrete Deck & Haunch (Noncomposite)	Wisconsin Barrier, Future Wearing Surface (Composite)	HI-93 Live Load (Composite)
Region A (0-84)	23605.3	53529.8	70696.2
Region B (84-104)	34639.8	67535.9	89807.8
Region C (104-120)	65426.6	104460.8	139158.7

Table E24-1.3-4
Section Properties Used to Generate Design Moments and Shears

E24-1.4 Compute Dead Load Effects

The girder must be designed to resist the dead load effects, as well as the other load effects. The dead load components consist of some dead loads that are resisted by the noncomposite section, as well as other dead loads that are resisted by the composite section. In addition, some dead loads are factored with the DC load factor and other dead loads are factored with the DW load factor. The following table summarizes the various dead load components that must be included in the design of a steel girder.

Dead Load Components		
Resisted by	**Type of Load Factor**	
	DC	**DW**
Noncomposite section	• Steel girder • Concrete deck • Concrete haunch • Stay-in-place deck forms • Miscellaneous dead load (including cross-frames, stiffeners, etc.)	
Composite section	• Concrete parapets	• Future wearing surface & utilities

Table E24-1.4-1
Dead Load Components

For the steel girder, the dead load per unit length varies due to the change in plate sizes. The moments and shears due to the weight of the steel girder can be computed using readily available analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

The dead load per unit length for Regions A, B, and C is calculated as follows:

$A_A = 49.75$ in^2 Region A (0 - 84 feet)(Table E24-1.3-1)

$A_B = 63.75$ in^2 Region B (84 - 104 feet)(Table E24-1.3-2)

$A_C = 100.50$ in^2 Region C (104 - 120 feet)(Table E24-1.3-3)

Weight of Girder per region:

$DL_A := W_S \cdot \frac{A_A}{12^2}$ $\boxed{DL_A = 0.169}$ klf

$DL_B := W_S \cdot \frac{A_B}{12^2}$ $\boxed{DL_B = 0.217}$ klf

$DL_C := W_S \cdot \frac{A_C}{12^2}$ $\boxed{DL_C = 0.342}$ klf

For the concrete deck, the dead load per unit length for an interior girder is computed as follows:

$w_c = 0.150$ kcf

$S = 10.00$ ft

$t_{deck} = 9.00$ in

$DL_{deck} := w_c \cdot S \cdot \frac{t_{deck}}{12}$ $\boxed{DL_{deck} = 1.125}$ kip/ft

For the concrete haunch, the dead load per unit length varies due to the change in top flange plate sizes. The moments and shears due to the concrete haunch are computed based on the actual, varying haunch thickness.

The haunch dead load per unit length for Region A, B, and C is calculated as follows:

$width_{flange} := 14$ in Top flange width is consistent in all three regions.

$t_{flangeA} := 0.75$ in Top flange thickness in Region A

$t_{flangeB} := 1.25$ in Top flange thickness in Region B

$t_{flangeC} := 2.5$ in Top flange thickness in Region C

$d_{haunch} = 3.75$ in Distance from top of web to bottom of deck as detailed in E24-1.1

$d_{hA} := d_{haunch} - t_{flangeA}$ $\boxed{d_{hA} = 3.00}$ in

$d_{hB} := d_{haunch} - t_{flangeB}$ $\boxed{d_{hB} = 2.50}$ in

$d_{hC} := d_{haunch} - t_{flangeC}$ $\boxed{d_{hC} = 1.25}$ in

$w_c = 0.150$ kcf

$DL_{hA} := \frac{width_{flange} \cdot d_{hA}}{12^2} \cdot w_c$ $\boxed{DL_{hA} = 0.044}$ klf

$DL_{hB} := \frac{width_{flange} \cdot d_{hB}}{12^2} \cdot w_c$ $\boxed{DL_{hB} = 0.036}$ klf

$$DL_{hC} := \frac{width_{flange} \cdot d_{hC}}{12^2} \cdot w_c$$ $\boxed{DL_{hC} = 0.018}$ klf

Total weight of deck and haunch per region:

$DL_{dhA} := DL_{deck} + DL_{hA}$ $\boxed{DL_{dhA} = 1.169}$ klf

$DL_{dhB} := DL_{deck} + DL_{hB}$ $\boxed{DL_{dhB} = 1.161}$ klf

$DL_{dhC} := DL_{deck} + DL_{hC}$ $\boxed{DL_{dhC} = 1.143}$ klf

For the miscellaneous dead load (including cross-frames, stiffeners, and other miscellaneous structural steel), the dead load per unit length is assumed to be as follows:

$DL_{misc} = 0.030$ kip/ft See E24-1.1

For the concrete parapets, the dead load per unit length is computed as follows, assuming that the superimposed dead load of the two parapets is distributed uniformly among all of the girders **LRFD [4.6.2.2.1]**:

$W_{par} = 0.464$ kip/ft (Type LF)

$N_b = 5$

$$DL_{par} := \frac{W_{par} \cdot 2}{N_b}$$ $\boxed{DL_{par} = 0.186}$ kip/ft

For the future wearing surface, the dead load per unit length is computed as follows, assuming that the superimposed dead load of the future wearing surface is distributed uniformly among all of the girders **LRFD [4.6.2.2.1]**:

$W_{fws} = 0.020$ ksf

$w_{roadway} = 44.0$ ft

$N_b = 5$

$$DL_{fws} := \frac{W_{fws} \cdot w_{roadway}}{N_b}$$ $\boxed{DL_{fws} = 0.176}$ kip/ft

Since the plate girder and its section properties are not uniform over the entire length of the bridge, an analysis software was used to compute the dead load moments and shears.

The following two tables present the unfactored dead load moments and shears, as computed by an analysis computer program. Since the bridge is symmetrical, the moments and shears in Span 2 are symmetrical to those in Span 1.

Dead Load Moments (Kip-feet)											
Dead Load Component	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Steel girder	0.0	71.7	119.1	142.1	140.7	114.9	64.8	-9.8	-112.1	-246.9	-430.4
Concrete deck & haunches	0.0	487.6	808.0	961.3	947.3	766.2	417.9	-97.6	-780.3	-1630.2	-2647.3
Other dead loads acting on girder alone	0.0	12.9	21.5	25.7	25.7	21.3	12.6	-0.4	-17.8	-39.5	-65.4
Concrete parapets	0.0	80.0	133.1	159.5	159.1	131.9	78.0	-2.8	-110.3	-244.6	-405.7
Future wearing surface	0.0	75.7	126.0	150.9	150.6	124.8	73.8	-2.6	-104.4	-231.5	-383.9

Table 24E1.4-2
Dead Load Moments

Dead Load Shears (Kips)											
Dead Load Component	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Steel girder	7.0	5.0	2.9	0.9	-1.1	-3.2	-5.2	-7.2	-9.8	-13.1	-17.5
Concrete deck & haunches	47.6	33.7	19.7	5.8	-8.1	-22.1	-36.0	-49.9	-63.9	-77.8	-91.7
Other dead loads acting on girder alone	1.3	0.9	0.5	0.2	-0.2	-0.5	-0.9	-1.3	-1.6	-2.0	-2.3
Concrete parapets	7.8	5.5	3.3	1.1	-1.1	-3.4	-5.6	-7.8	-10.1	-12.3	-14.5
Future wearing surface	7.4	5.2	3.1	1.0	-1.1	-3.2	-5.3	-7.4	-9.5	-11.6	-13.8

Table 24E1.4-3
Dead Load Shears

E24-1.5 Compute Live Load Effects

The girder must also be designed to resist the live load effects **LRFD [3.6.1.2]**. The live load consists of an HL-93 loading. Similar to the dead load, the live load moments and shears for an HL-93 loading were obtained from an analysis computer program.

Based on Table E24-1.1-3, for all limit states other than fatigue and fracture, the dynamic load allowance, IM, is as follows **LRFD [3.6.2.1]**:

$$IM := 0.33$$

The live load distribution factors for moment for an interior girder are computed as follows **LRFD [4.6.2.2.2]**:

First, the longitudinal stiffness parameter, K_g, must be computed **LRFD [4.6.2.2.1]**:

$$K_g := n \cdot \left(I + A \cdot e_g^{\,2}\right)$$

Where:

I = Moment of inertia of beam (in^4)

A = Area of stringer, beam, or girder (in^2)

e_g = Distance between the centers of gravity of the basic beam and deck (in)

Longitudinal Stiffness Parameter, K_g				
	Region A (Pos. Mom.)	**Region B (Intermediate)**	**Region C (At Pier)**	**Weighted Average ***
Length (Feet)	84	20	16	
n	8	8	8	
I (Inches^4)	23,605.3	34,639.8	65,426.6	
A (Inches^2)	49.750	63.750	100.500	
e_g (Inches)	35.978	35.777	36.032	
K_g (Inches^4)	704,020	929,915	1,567,250	856,767

Table E24-1.5-1
Longitudinal Stiffness Parameter

After the longitudinal stiffness parameter is computed, **LRFD [Table 4.6.2.2.1-1]** is used to find the letter corresponding with the superstructure cross section. The letter corresponding with the superstructure cross section in this design example is "a."

If the superstructure cross section does not correspond with any of the cross sections illustrated in **LRFD [Table 4.6.2.2.1-1]**, then the bridge should be analyzed as presented in **LRFD [4.6.3]**.

Based on cross section "a", **LRFD [Table 4.6.2.2.2b-1 & Table 4.6.2.2.3a-1]** are used to compute the distribution factors for moment and shear, respectively.

Check the range of applicability as follows **LRFD [Table 4.6.2.2.2b-1]**:

$\boxed{3.5 \le S \le 16.0}$

Where:

S = Spacing of beams or webs (ft)

$S = 10.00$ ft OK

$\boxed{4.5 \le t_s \le 12.0}$

Where:

t_s = Depth of concrete slab (in)

$t_s = 8.5$ in OK

$\boxed{20 \le L \le 240}$

Where:

L = Span of beam (ft)

$L := 120$ ft OK

$\boxed{N_b \ge 4}$

Where:

N_b = Number of beams, stringers, or girders

$N_b = 5.00$ OK

$\boxed{10000 \le K_g \le 7000000}$

$K_g := 856767$ in^4 OK

For one design lane loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [4.6.2.2.2b-1]**:

$$g_{int\_moment\_1} := 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0L \cdot t_s^{\,3}}\right)^{0.1}$$

$\boxed{g_{int\_moment\_1} = 0.473}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [Table 4.6.2.2.2b-1]**:

$$g_{int\_moment\_2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0 \cdot L \cdot t_s^{\,3}}\right)^{0.1}$$

$\boxed{g_{int\_moment\_2} = 0.700}$ lanes

The live load distribution factors for shear for an interior girder are computed in a similar manner. The range of applicability is similar to that for moment **LRFD [Table 4.6.2.2.3a-1]**.

For one design lane loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{int\_shear\_1} := 0.36 + \frac{S}{25.0}$$

$\boxed{g_{int\_shear\_1} = 0.760}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{int\_shear\_2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$\boxed{g_{int\_shear\_2} = 0.952}$ lanes

Since this bridge has no skew, the skew correction factor does not need to be considered for this design example **LRFD [4.6.2.2.2e & 4.6.2.2.3c]**.

This design example is based on an interior girder. However, for illustrative purposes, the live load distribution factors for an exterior girder are computed below, as follows **LRFD [4.6.2.2.2]**:

The distance, d_e, is defined as the distance between the web centerline of the exterior girder and the interior edge of the curb. For this design example, based on Figure E24-1.1-2:

$d_e := S_{overhang} - 1.25$ ft

Check the range of applicability as follows **LRFD [Table 4.6.2.2.2d-1]**:

$-1.0 \le d_e \le 5.5$

$\boxed{d_e = 2.00}$ ft OK

For one design lane loaded, the distribution of live load per lane for moment in exterior beams is computed using the lever rule, as follows:

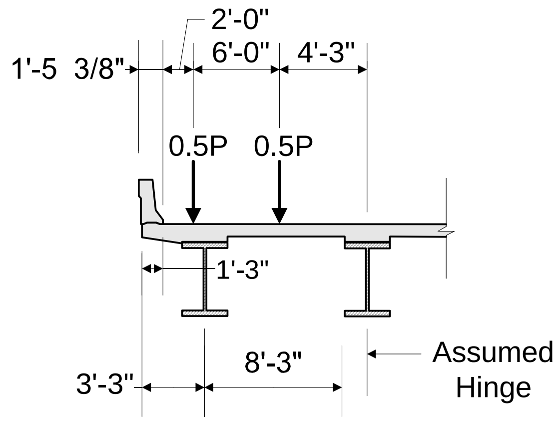


Figure E24-1.5-1
Lever Rule

$$x_1 := S - 6 + (d_e - 2) \qquad x_2 := S + (d_e - 2)$$

$$g_{ext\_moment\_1} := \frac{(0.5)\cdot(x_1) + (0.5)\cdot(x_2)}{S} \qquad \boxed{g_{ext\_moment\_1} = 0.700} \text{ lanes}$$

$$mpf := 1.20$$

$$g_{ext\_moment\_1} := g_{ext\_moment\_1} \cdot mpf \qquad \boxed{g_{ext\_moment\_1} = 0.840}$$

lanes
(for strength limit state)

For two or more design lanes loaded, the distribution of live load per lane for moment in exterior beams is as follows **LRFD [Table 4.6.2.2.2d-1]**:

The correction factor for distribution, e, is computed as follows:

$$e := 0.77 + \frac{d_e}{9.1} \qquad \boxed{e = 0.990}$$

$$g_{ext\_moment\_2} := e \cdot g_{int\_moment\_2} \qquad \boxed{g_{ext\_moment\_2} = 0.693} \text{ lanes}$$

The live load distribution factors for shear for an exterior girder are computed in a similar manner. The range of applicability is similar to that for moment **LRFD [Table 4.6.2.2.3.b-1]**.

For one design lane loaded, the distribution of live load per lane for shear in exterior beams is computed using the lever rule, as illustrated in Figure E24-1.5-1 and as follows:

$$g_{ext\_shear\_1} := \frac{(0.5)\cdot(x_1) + (0.5)\cdot(x_2)}{S} \qquad \boxed{g_{ext\_shear\_1} = 0.700} \text{ lanes}$$

$$g_{ext\_shear\_1} := g_{ext\_shear\_1} \cdot mpf \qquad \boxed{g_{ext\_shear\_1} = 0.840}$$

lanes
(for strength limit state)

For two or more design lanes loaded, the distribution of live load per lane for shear in exterior beams is as follows **LRFD [Table 4.6.2.2.3b-1]**:

$e := 0.6 + \frac{d_e}{10}$ $\boxed{e = 0.800}$

$g_{ext\_shear\_2} := e \cdot g_{int\_shear\_2}$ $\boxed{g_{ext\_shear\_2} = 0.761}$ lanes

In beam-slab bridge cross-sections with diaphragms or cross-frames, the distribution factor for the exterior beam can not be taken to be less than that which would be obtained by assuming that the cross-section deflects and rotates as a rigid cross-section. **LRFD [C4.6.2.2.2d]** provides one approximate approach to satisfy this requirement. The multiple presence factor provisions of **LRFD [3.6.1.1.2]** must be applied when this equation is used. This is not shown here since an interior girder is being designed.

Since this bridge has no skew, the skew correction factor does not need to be considered for this design example **LRFD [4.6.2.2.2e & 4.6.2.2.3c]**.

The controlling distribution factors for moment and shear for the interior girder are given below.

Interior Girder Distribution Factors		
	Moment DF	**Shear DF**
One Lane	0.473	0.760
Two or More Lanes	0.700	0.952

Table E24-1.5-2
Summary of Interior Girder Distribution Factors

The following table presents the unfactored maximum positive and negative live load moments and shears for HL-93 live loading for interior beams, as computed using an analysis computer program. These values include the controlling live load distribution factor given above for two or more lanes, and they also include dynamic load allowance. Since the bridge is symmetrical, the moments and shears in Span 2 are symmetrical to those in Span 1.

Live Load Effects (for Interior Beams)											
Live Load Effect	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Maximum positive moment (K-ft)	0.0	858.4	1449.2	1792.8	1923.5	1858.2	1616.5	1207.4	684.9	264.2	0.0
Maximum negative moment (K-ft)	0.0	-143.2	-286.4	-429.6	-572.8	-716.0	-859.2	-1002.4	-1165.9	-1724.3	-2571.5
Maximum positive shear (kips)	112.8	94.1	76.7	60.8	46.5	34.0	23.3	14.5	7.6	3.0	0.0
Maximum negative shear (kips)	-16.2	-16.7	-22.8	-36.3	-50.8	-65.6	-80.4	-94.9	-108.8	-122.1	-134.9

Table 24E1.5-2
Live Load Effects

The design live load values for HL-93 loading, as presented in the previous table, are computed based on the product of the live load effect per lane and live load distribution factor. These values also include the effects of dynamic load allowance. However, it is important to note that the dynamic load allowance is applied only to the design truck or tandem. The dynamic load allowance is not applied to pedestrian loads or to the design lane load **LRFD [3.6.1, 3.6.2, 4.6.2.2]**.

E24-1.6 Combine Load Effects

After the load factors and load combinations have been established (see E24-1.1), the section properties have been computed (see E24-1.3), and all of the load effects have been computed (see E24-1.4 and E24-1.5), the force effects must be combined for each of the applicable limit states.

For this design example, η equals 1.00 **LFRD[1.3]**. (For more detailed information about η, refer to E24-1.1.)

The maximum positive moment (located at 0.4L) for the Strength I Limit State is computed as follows **LRFD [3.4.1]**:

$LF_{DC} := 1.25$

$M_{DC} := 140.7 + 947.3 + 25.7 + 159.1$ $\boxed{M_{DC} = 1272.8}$ kip-ft

$LF_{DW} := 1.50$

$M_{DW} := 150.6$ kip-ft

$LF_{LL} := 1.75$

$M_{LL} := 1923.5$

$M_{total} := LF_{DC} \cdot M_{DC} + LF_{DW} \cdot M_{DW} + LF_{LL} \cdot M_{LL}$ $\boxed{M_{total} = 5183.0}$ kip-ft

Similarly, the maximum stress in the top of the girder due to positive moment (located at 0.4L) for the Strength I Limit State is computed as follows:

Noncomposite dead load:

$M_{noncompDL} := 140.7 + 947.3 + 25.7$ $\boxed{M_{noncompDL} = 1113.70}$ kip-ft

$S_{topgdr} := 821.7$ in^3

$$f_{noncompDL} := \frac{-M_{noncompDL} \cdot (12)}{S_{topgdr}}$$ $\boxed{f_{noncompDL} = -16.26}$ ksi

Parapet dead load (composite):

$M_{parapet} := 159.1$ kip-ft

$S_{topgdr} := 4404.7$ in^3

$$f_{parapet} := \frac{-M_{parapet} \cdot (12)}{S_{topgdr}} \qquad \boxed{f_{parapet} = -0.43} \quad \text{ksi}$$

Future wearing surface dead load (composite):

$M_{fws} := 150.6$ kip-ft

$S_{topgdr} := 4404.7$ in^3

$$f_{fws} := \frac{-M_{fws} \cdot (12)}{S_{topgdr}} \qquad \boxed{f_{fws} = -0.41} \quad \text{ksi}$$

Live load (HL-93) and dynamic load allowance:

$M_{LL} = 1923.50$ kip-ft

$S_{topgdr} := 24820.6$ in^3

$$f_{LL} := \frac{-M_{LL} \cdot (12)}{S_{topgdr}} \qquad \boxed{f_{LL} = -0.93} \quad \text{ksi}$$

Multiplying the above stresses by their respective load factors and adding the products results in the following combined stress for the Strength I Limit State **LRFD [3.4.1]**:

$$f_{Str} := (LF_{DC} \cdot f_{noncompDL}) + (LF_{DC} \cdot f_{parapet}) + (LF_{DW} \cdot f_{fws}) + (LF_{LL} \cdot f_{LL})$$

$$\boxed{f_{Str} = -23.12} \quad \text{ksi}$$

Similarly, all of the combined moments, shears, and flexural stresses can be computed at the controlling locations. A summary of those combined load effects for an interior beam is presented in the following three tables, summarizing the results obtained using the procedures demonstrated in the above computations.

Combined Effects at Location of Maximum Positive Moment				
Summary of Unfactored Values:				
Loading	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**$f_{topslab}$ (ksi)**
Noncomposite DL	1102.0	15.07	-16.09	0.00
Parapet DL	136.9	1.35	-0.41	-0.04
FWS DL	155.4	1.53	-0.47	-0.05
LL - HL-93	1916.6	17.27	-1.18	-0.62
LL - Fatigue Range	871.4	7.85	-0.54	-0.28
Summary of Factored Values:				
Limit State	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**$f_{topslab}$ (ksi)**
Strength I	5135.8	53.03	-23.40	-1.22
Service II	3885.9	40.39	-18.51	-0.91
Fatigue I	653.5	5.89	-0.40	-0.21

Table E24-1.6-1
Combined Effects at Location of Maximum Positive Moment

As shown in the above table, the Strength I Limit State elastic stress in the bottom of the girder exceeds the girder yield stress. However, for this design example, this value is not used because of the local yielding that is permitted to occur at this section at the strength limit state.

Combined Effects at Location of Maximum Negative Moment				
Summary of Unfactored Values (Assuming Concrete Not Effective):				
Loading	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Noncomposite DL	-3073.2	-16.19	17.21	0.00
Parapet DL	-327.5	-1.66	1.52	1.82
FWS DL	-371.9	-1.88	1.73	2.06
LL - HL-93	-2414.2	-12.21	11.23	13.40
Summary of Unfactored Values (Assuming Concrete Effective):				
Loading	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Noncomposite DL	-3073.2	-16.19	17.21	0.00
Parapet DL	-327.5	-1.49	0.79	0.08
FWS DL	-371.9	-1.70	0.90	0.09
LL - HL-93	-2414.2	-10.23	2.39	0.56
LL - Fatigue Range	-481.4	-2.04	0.48	0.11
Summary of Factored Values:				
Limit State	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Strength I *	-9033.6	-46.50	45.66	28.82
Service II **	-6911.1	-32.67	22.01	0.89
Fatigue I **	-361.0	-1.53	0.36	0.08

Legend:

* Strength I Limit State stresses are based on section properties assuming the deck concrete is not effective, and f_{deck} is the stress in the deck reinforcing steel.

** Service II and Fatigue I Limit State stresses are based on section properties assuming the deck concrete is effective, and f_{deck} is the stress in the deck concrete.

Table E24-1.6-2

Combined Effects at Location of Maximum Negative Moment

Combined Effects at Location of Maximum Shear	
Summary of Unfactored Values:	
Loading	**Shear (kips)**
Noncomposite DL	108.9
Parapet DL	12.0
FWS DL	13.7
LL - HL-93	132.0
LL - Fatigue Range	56.5
Summary of Factored Values:	
Limit State	**Shear (kips)**
Strength I	402.7
Service II	306.2
Fatigue I	42.4

Table E24-1.6-3
Combined Effects at Location of Maximum Shear

Envelopes of the factored Strength I moments and shears are presented in the following two figures. Maximum and minimum values are presented. As mentioned previously, all remaining design computations in this example are based on the interior girder. The basic approach illustrated in the subsequent design calculations applies equally to the exterior and interior girders (with some exceptions noted) once the load effects in each girder have been determined.

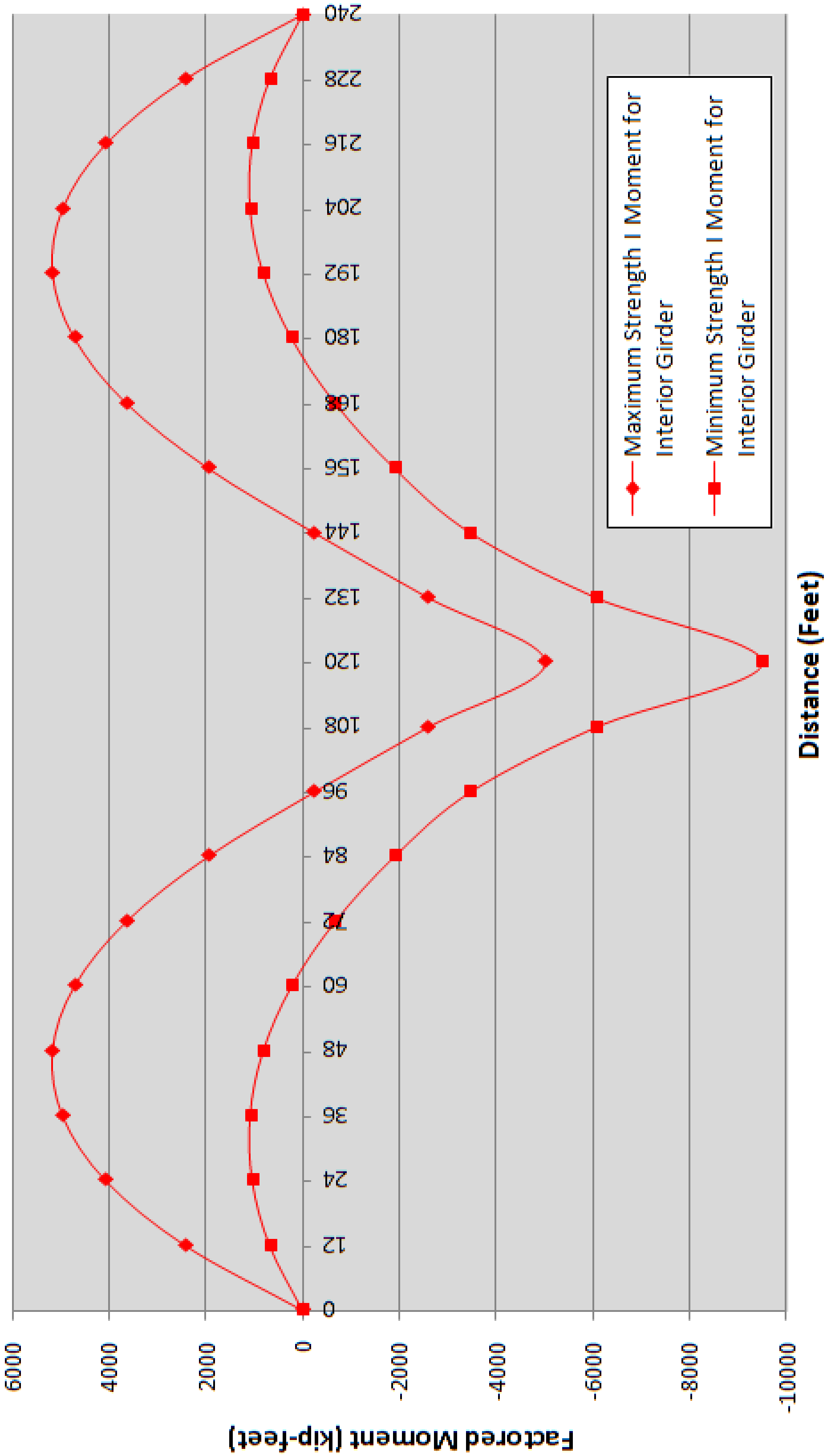


Figure 24E1.6-1
Envelope of Strength I Moments

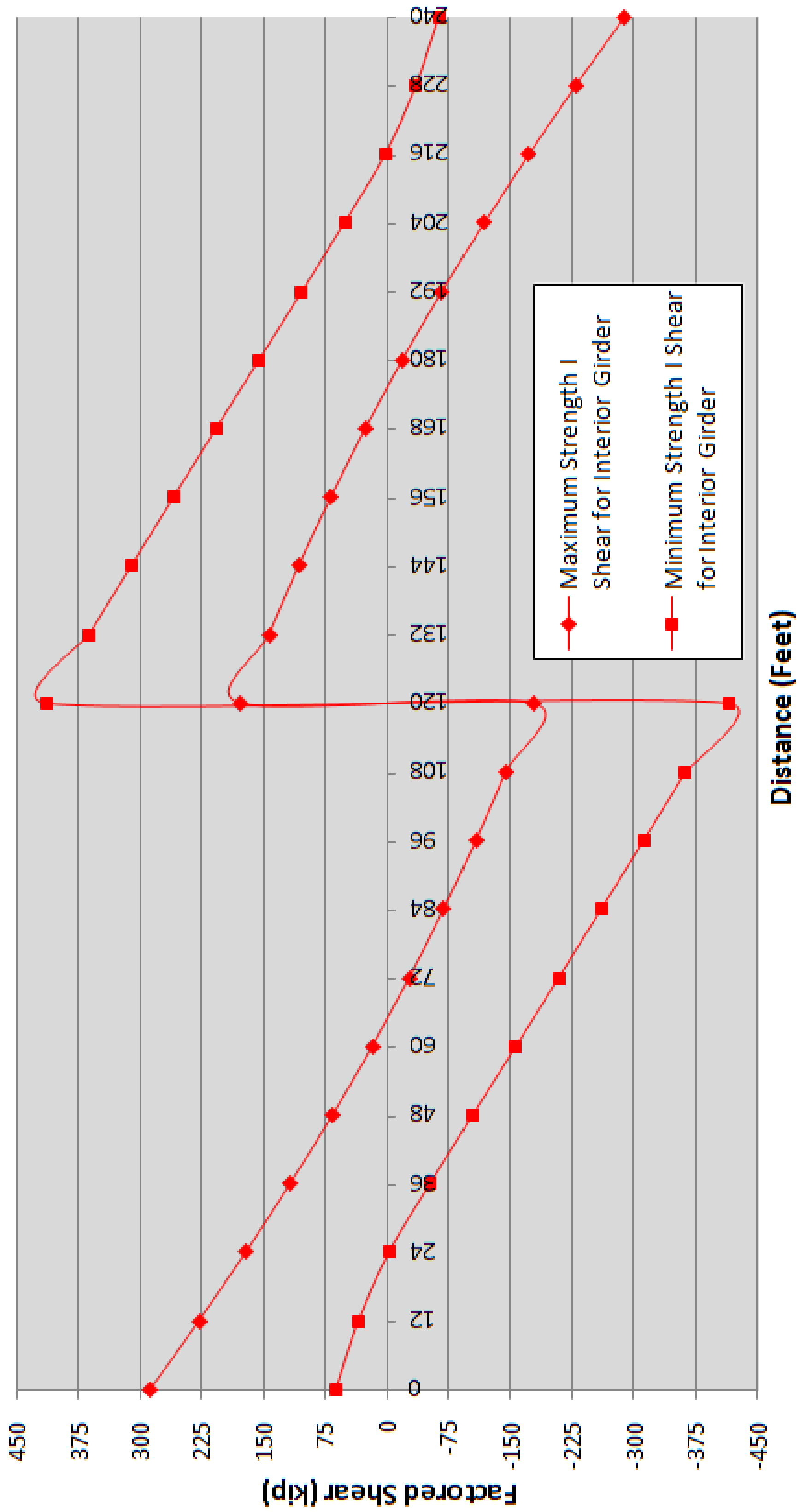


Figure 24E1.6-2
Envelope of Strength I Shears

Two design sections will be checked for illustrative purposes. First, all specification checks will be performed for the location of maximum positive moment, which is at 0.4L in Span 1. Second, all specification checks for these same design steps will be performed for the location of maximum negative moment and maximum shear, which is at the pier.

The following specification checks are for the location of maximum positive moment, which is at 0.4L in Span 1, as shown in Figure E24-1.6-3.

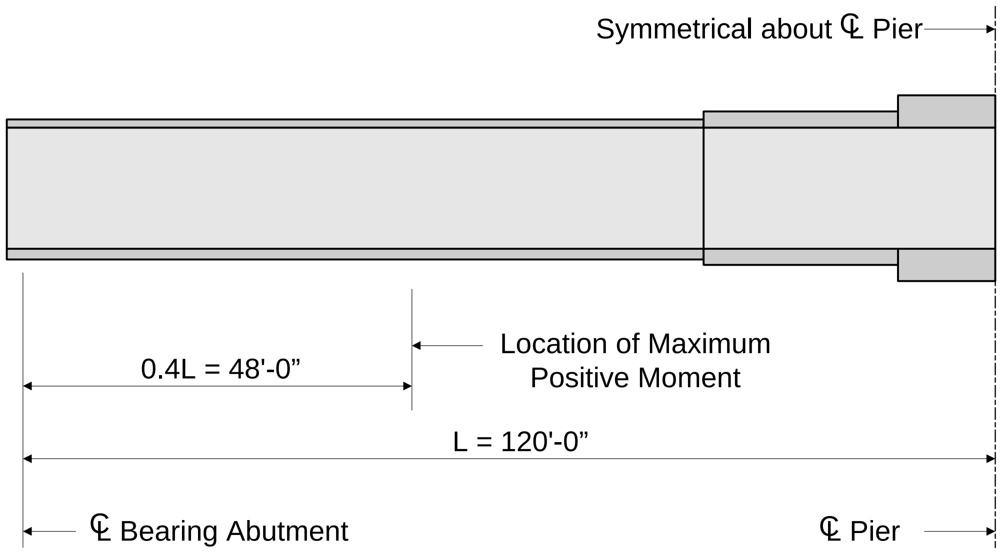


Figure E24-1.6-3
Location of Maximum Positive Moment

E24-1.7 Check Section Proportion Limits - Positive Moment Region

Several checks are required to ensure that the proportions of the trial girder section are within specified limits **LRFD [6.10.2]**.

The first section proportion check relates to the web slenderness **LRFD [6.10.2.1]**. For a section without longitudinal stiffeners, the web must be proportioned such that:

$$\frac{D}{t_w} \leq 150$$

Where:

D	= Clear distance between flanges (in)
t_w	= Web thickness (in)

$D := 54$ in

$t_w := 0.50$ in

$$\boxed{\frac{D}{t_w} = 108.00} \quad \text{OK}$$

The second set of section proportion checks relate to the general proportions of the section **LRFD [6.10.2.2]**. The compression and tension flanges must be proportioned such that:

$$\frac{b_f}{2 \cdot t_f} \le 12.0$$

Where:

b_f = Full width of the flange (in)

t_f = Flange thickness (in)

$b_f := 14$ $\quad t_f := 0.75$ $\quad \boxed{\frac{b_f}{2 \cdot t_f} = 9.33}$ OK

$b_f \ge \frac{D}{6}$ $\quad \boxed{\frac{D}{6} = 9.00}$ in OK

$t_f \ge 1.1 \cdot t_w$ $\quad \boxed{1.1t_w = 0.55}$ in OK

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10$$

Where:

I_{yc} = moment of inertia of the compression flange of a steel section about the vertical axis in the plane of the web (in^4)

I_{yt} = moment of inertia of the tension flange of a steel section about the vertical axis in the plane of the web (in^4)

$I_{yc} := \frac{0.75 \cdot 14^3}{12}$ $\quad \boxed{I_{yc} = 171.50}$ in^4

$I_{yt} := \frac{0.875 \cdot 14^3}{12}$ $\quad \boxed{I_{yt} = 200.08}$ in^4

$\boxed{\frac{I_{yc}}{I_{yt}} = 0.857}$ OK

E24-1.8 Compute Plastic Moment Capacity - Positive Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**.

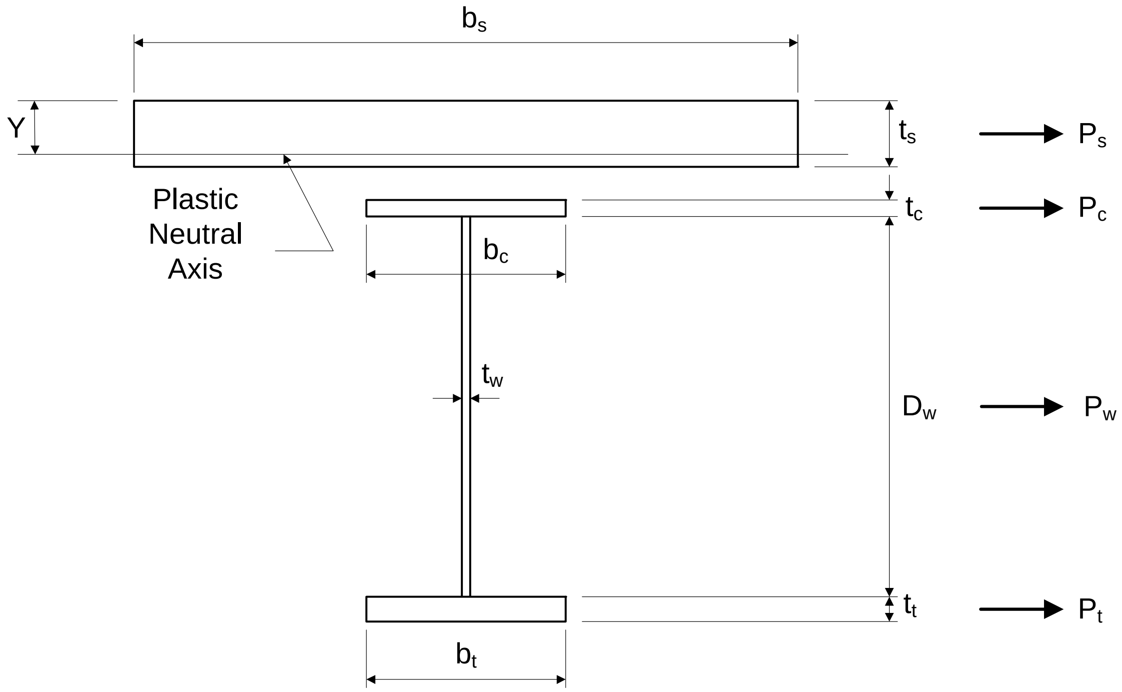


Figure E24-1.8-1
Computation of Plastic Moment Capacity for Positive Bending Sections

For the tension flange:

$P_t := F_{yt} \cdot b_t \cdot t_t$

Where:

F_{yt}	= Specified minimum yield strength of a tension flange (ksi)
b_t	= Full width of the tension flange (in)
t_t	= Thickness of tension flange (in)

$F_{yt} := 50$ ksi

$b_t := 14$ in

$t_t := 0.875$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 613}$ kips

For the web:

$P_w := F_{yw} \cdot D \cdot t_w$

Where:

	F_{yw}	= Specified minimum yield strength of a web (ksi)

$F_{yw} = 50$ ksi

$D = 54$ in

$t_w = 0.50$ in

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

For the compression flange:

$P_c := F_{yc} \cdot b_c \cdot t_c$

Where:

F_{yc}	= Specified minimum yield strength of a compression flange (ksi)
b_c	= Full width of the compression flange (in)
t_c	= Thickness of compression flange (in)

$F_{yc} := 50$ ksi

$b_c := 14$ in

$t_c := 0.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 525}$ kips

For the slab:

$P_s := 0.85 \cdot f'_c \cdot b_s \cdot t_s$

Where:

b_s	= Effective width of concrete deck (in)
t_s	= Thickness of concrete deck (in)

$f'_c = 4.00$ ksi

$b_s := 120$ in

$t_s = 8.5$ in

$P_s := 0.85 \cdot f'_c \cdot b_s \cdot t_s$ $\boxed{P_s = 3468}$ kips

The forces in the longitudinal reinforcement may be conservatively neglected in regions of positive flexure.

Check the location of the plastic neutral axis, as follows:

$\boxed{P_t + P_w = 1963}$ kips $\boxed{P_c + P_s = 3993}$ kips

$\boxed{P_t + P_w + P_c = 2488}$ kips $\boxed{P_s = 3468}$ kips

Since $P_t + P_w + P_c < P_s$, the plastic neutral axis is located within the slab **LRFD [Table D6.1-1]**. Since the slab reinforcement is being neglected in regions of positive flexure, Case III, V, or VII can be used. All three cases yield the same results with the reinforcement terms P_{rt} and P_{rb} set equal to zero.

$$Y := (t_s) \cdot \left(\frac{P_c + P_w + P_t}{P_s} \right)$$ $\boxed{Y = 6.10}$ in

Check that the position of the plastic neutral axis, as computed above, results in an equilibrium condition in which there is no net axial force.

$Compression := 0.85 \cdot f'_c \cdot b_s \cdot Y$ $\boxed{Compression = 2488}$ kips

$Tension := P_t + P_w + P_c$ $\boxed{Tension = 2488}$ kips OK

The plastic moment, M_p, is computed as follows, where d is the distance from an element force (or element neutral axis) to the plastic neutral axis **LRFD [Table D6.1-1]**:

$$d_c := \frac{-t_c}{2} + 3.75 + t_s - Y$$ $\boxed{d_c = 5.78}$ in

$$d_w := \frac{D}{2} + 3.75 + t_s - Y$$ $\boxed{d_w = 33.15}$ in

$$d_t := \frac{t_t}{2} + D + 3.75 + t_s - Y$$ $\boxed{d_t = 60.59}$ in

$$M_p := \frac{\frac{Y^2 \cdot P_s}{2 \cdot t_s} + \left(P_c \cdot d_c + P_w \cdot d_w + P_t \cdot d_t\right)}{12}$$ $\boxed{M_p = 7707}$ kip-ft

E24-1.9 Determine if Section is Compact or Noncompact - Positive Moment Region

Since the section is in a straight bridge, the next step in the design process is to determine if the section is compact or noncompact. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

If the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the compact-section web slenderness provisions, as follows **LRFD [6.10.6.2.2]**:

$$\frac{2 \cdot D_{cp}}{t_w} \leq 3.76 \cdot \sqrt{\frac{E}{F_{yc}}}$$

Where:

D_{cp} = Depth of web in compression at the plastic moment (in)

Since the plastic neutral axis is located within the slab,

$D_{cp} := 0$ in

Therefore the web is deemed compact. Since this is a composite section in positive flexure and there are no holes in the tension flange at this section, the flexural resistance is computed as defined by the composite compact-section positive flexural resistance provisions of **LRFD [6.10.7.1.2]**.

E24-1.10 Design for Flexure - Strength Limit State - Positive Moment Region

Since the section was determined to be compact, and since it is a composite section in the positive moment region with no holes in the tension flange, the flexural resistance is computed in accordance with **LRFD [6.10.7.1.2]**.

$$M_n := 1.3 \cdot R_h \cdot M_y$$

Where:

R_h = Hybrid factor

M_y = Yield Moment (kip-in)

All design sections of this girder are homogenous. That is, the same structural steel is used for the top flange, the web, and the bottom flange. Therefore, the hybrid factor, R_h, is as follows **LRFD [6.10.1.10.1]**:

$R_h := 1.0$

The yield moment, M_y, is computed as follows **LRFD [Appendix D6.2.2]**:

$$F_y := \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

Where:

M_{D1} = Bending moment caused by the factored permanent load applied before the concrete deck has hardened or is made composite (kip-in)

S_{NC} = Noncomposite elastic section modulus (in^3)

M_{D2} = Bending moment caused by the factored permanent load applied to the long-term composite section (kip-in)

S_{LT} = Long-term composite elastic section modulus (in^3)

M_{AD} = Additional bending moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange (kip-in)

S_{ST} = Short-term composite elastic section modulus (in^3)

$$M_y := M_{D1} + M_{D2} + M_{AD}$$

$F_y := 50$ ksi

$M_{D1} := (1.25 \cdot 1113.7)$ $\boxed{M_{D1} = 1392}$ kip-ft

$M_{D2} := (1.25 \cdot 159.1) + (1.50 \cdot 150.6)$ $\boxed{M_{D2} = 425}$ kip-ft

For the bottom flange:

$S_{NC} := 877.6$ in^3

$S_{LT} := 1231.4$ in^3

$S_{ST} := 1339.5$ in^3

$$M_{AD} := \left[\frac{S_{ST}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT}}{12^3}}\right)\right] \qquad \boxed{M_{AD} = 2994} \text{ kip-ft}$$

$M_{ybot} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ybot} = 4811}$ kip-ft

For the top flange:

$S_{NC} := 821.7$ in^3

$S_{LT} := 4404.7$ in^3

$S_{ST} := 24820.6$ in^3

$$M_{AD} := \frac{S_{ST}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT}}{12^3}}\right) \qquad \boxed{M_{AD} = 58974} \text{ kip-ft}$$

$M_{ytop} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ytop} = 60791}$ kip-ft

The yield moment, M_y, is the lesser value computed for both flanges. Therefore, M_y is determined as follows **LRFD [Appendix D6.2.2]**:

$M_y := \min(M_{ybot}, M_{ytop})$ $\boxed{M_y = 4811}$ kip-ft

Therefore, for the positive moment region of this design example, the nominal flexural resistance is computed as follows **LRFD [6.10.7.1.2]**:

$D_p \le 0.1D_t$

$D_p := Y$ $\boxed{D_p = 6.10}$ in

$D_t := 0.875 + 54 + .75 + 3 + 8.5$ $\boxed{D_t = 67.13}$ in

$\boxed{0.1 \cdot D_t = 6.713}$ in OK

Therefore

$M_n := M_p$ $\boxed{M_n = 7707}$ kip-ft

Since this is neither a simple span nor a continuous span where the span and the sections in the negative-flexure region over the interior supports satisfy the special conditions outlined at the end of **LRFD [6.10.7.1.2]**, the nominal flexural resistance of the section must not exceed the following:

$M_n := 1.3 \cdot R_h \cdot M_y$ $\boxed{M_n = 6255}$ kip-ft

The ductility requirement is checked as follows **LRFD [6.10.7.3]**:

$D_p \le 0.42D_t$

Where:

D_p = Distance from top of the concrete deck to the neutral axis of the composite section at the plastic moment (in)

D_t = Total depth of the composite section (in)

$\boxed{0.42 \cdot D_t = 28.19}$ in OK

The factored flexural resistance, M_r, is computed as follows (note that since there is no curvature, skew and wind load is not considered under the Strength I load combination, the flange lateral bending stress is taken as zero in this case **LRFD [6.10.7.1.1]**:

$$M_u + \frac{1}{3}(0) \le \phi_f M_n$$

Where:

M_u = Moment due to the factored loads (kip-in)

M_n = Nominal flexural resistance of a section (kip-in)

$\phi_f := 1.00$

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 6255}$ kip-ft

The positive flexural resistance at this design section is checked as follows:

$\Sigma\eta_i \cdot \gamma_i \cdot Q_i \le R_r$

or in this case:

$\Sigma\eta \cdot \gamma \cdot M_u \le M_r$

$\eta := 1.00$

As computed in E24-1.6,

$\Sigma\gamma \cdot M_u := 5183$ kip-ft

Therefore

$\Sigma\eta \cdot \gamma \cdot M_u := 5183$ kip-ft

$\boxed{M_r = 6255}$ kip-ft OK

E24-1.11 Design for Shear - Positive Moment Region

Shear must be checked at each section of the girder **LRFD [6.10.9]**. However, shear is minimal at the location of maximum positive moment, and it is maximum at the pier.

Therefore, for this design example, the required shear design computations will be presented later for the girder design section at the pier.

It should be noted that in end panels, the shear is limited to either the shear yield or shear buckling in order to provide an anchor for the tension field in adjacent interior panels. Tension field is not allowed in end panels. The design procedure for shear in the end panel is presented in **LRFD [6.10.9.3.3c]**.

E24-1.12 Design Transverse Intermediate Stiffeners - Positive Moment Region

As stated above, shear is minimal at the location of maximum positive moment but is maximum at the pier. Therefore, the required design computations for transverse intermediate stiffeners will be presented later for the girder design section at the pier **LRFD [6.10.11.1]**.

E24-1.13 Design for Flexure - Fatigue and Fracture Limit State - Positive Moment Region

Load-induced fatigue must be considered in a plate girder design **LRFD [6.6.1]**.

For this design example, fatigue will be checked for the fillet-welded connection of a transverse intermediate stiffener serving as a cross-frame connection plate to the girder at the location of maximum positive moment. This detail corresponds to Description 4.1 in **LRFD [Table 6.6.1.2.3-1]**, and it is classified as Detail Category C'. The fatigue detail at the inner fiber of the tension flange, where the transverse intermediate stiffener is welded to the flange, is subject to a net tensile stress by inspection. However, for simplicity, the computations will conservatively compute the fatigue stress at the outer fiber of the tension flange.

The fatigue detail being investigated in this design example is illustrated in the following figure:

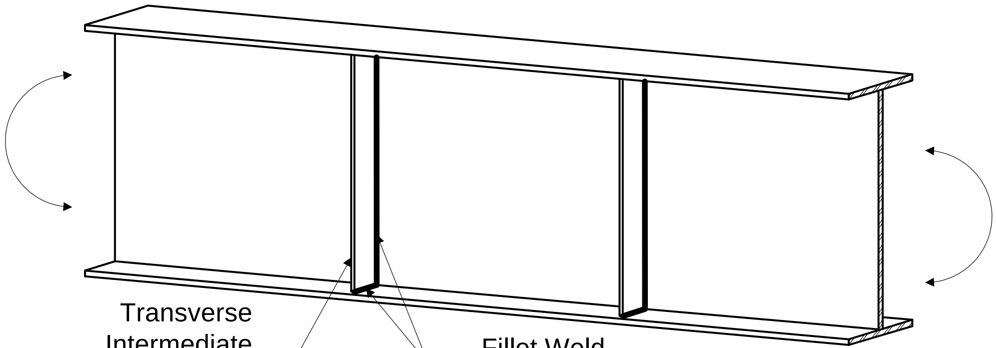


Figure E24-1.13-1
Load-Induced Fatigue Detail

The nominal fatigue resistance is computed as follows **LRFD [6.6.1.2.5]**:

NOTE: WisDOT policy is to design for infinite fatigue life (ADTT not considered) and use Fatigue I limit state.

$\Delta F_n := \Delta F_{TH}$

Where:

ΔF_{TH} = Constant-amplitude fatigue threshold **LRFD [Table 6.6.1.2.5-3]** (ksi)

$\Delta F_{TH} := 12.00$ ksi

$\Delta F_n = \blacksquare \cdot 12.00$ ksi

The factored fatigue stress range in the outer fiber base metal at the weld at the location of maximum positive moment was previously computed in Table E24-1.6-1, as follows:

$f_{botgdr} := 9.90$ ksi

$f_{botgdr} \leq \Delta F_n$ OK

In addition to the above fatigue detail check, a special fatigue requirement for webs must also be checked **LRFD [6.10.6]**. These calculations will be presented later for the girder design section at the pier [E24-1.23].

E24-1.14 Design for Flexure - Service Limit State - Positive Moment Region

The girder must be checked for service limit state control of permanent deflection **LRFD [6.10.4.2]**. This check is intended to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. The Service II load combination is used for this check.

The stresses for steel flanges of composite sections must satisfy the following requirements **LRFD [6.10.4.2.2]**:

Top flange:

$$f_f \leq 0.95R_h \cdot F_{yf}$$

Bottom flange

$$f_f + \frac{f_l}{2} \leq 0.95R_h \cdot F_{yf}$$

Since there is no curvature and no discontinuous diaphragm lines in conjunction with skews exceeding 20 degrees, f_l is taken equal to zero at the service limit state in this case. The factored Service II flexural stress was previously computed in Table E24-1.6-1 as follows:

$f_{botgdr} := 40.65$ ksi

$f_{topgdr} := -18.32$ ksi

$\boxed{0.95 \cdot R_h \cdot F_{yf} = 47.50}$ ksi OK

As indicated in **LRFD [6.10.4.2.2]**, the web bend buckling check at the service limit state must be checked for all sections according to equation 6.10.4.2.2-4 with the execption of composite sections in positive flexure that meet the requirement of **LRFD [6.10.2.1.1]** (D/tw ≤ 150). Since $\frac{D}{t_w} = 108$ [E24-1.7], equation 6.10.4.2.2-4 does not need to be considered for this location.

In addition to the check for service limit state control of permanent deflection, the girder can also be checked for live load deflection **LRFD [2.5.2.6.2]**. Although this check is optional for a concrete deck on steel girders, it is included in this design example.

Using an analysis computer program, the maximum live load deflection is computed to be the following:

$\Delta_{max} := 1.14$ in

This maximum live load deflection is computed based on the following:

1. All design lanes are loaded.
2. All supporting components are assumed to deflect equally.
3. For composite design, the design cross section includes the entire width of the roadway.
4. The number and position of loaded lanes is selected to provide the worst effect.
5. The live load portion of Service I Limit State is used.
6. Dynamic load allowance is included.
7. The live load is taken from **LRFD [3.6.1.3.2]**.

As recommended in LRFD [2.5.2.6.2] for "vehicular load, general", the deflection limit is as follows:

$Span := 120$ ft

$\Delta_{allowable} := \left(\frac{Span}{800}\right)\cdot(12)$ $\boxed{\Delta_{allowable} = 1.80}$ in OK

E24-1.15 Design for Flexure - Constructibility Check - Positive Moment Region

The girder must also be checked for flexure during construction **LRFD [6.10.3.2]**. The girder has already been checked in its final condition when it behaves as a composite section. The constructibility must also be checked for the girder prior to the hardening of the concrete deck when the girder behaves as a noncomposite section.

As previously stated, a deck pouring sequence will not be considered in this design example. However, it is required to consider the effects of the deck pouring sequence in an actual design because it will often control the design of the top flange and the cross-frame spacing in the positive moment regions of composite girders. The calculations illustrated below, which are based on the final noncomposite dead load moments after the sequential placement is complete would be employed to check the girder for the critical actions resulting from the deck pouring sequence. For an exterior girder, deck overhang effects must also be considered according to **LRFD [6.10.3.4]**. Since an interior girder is designed in this example, those effects are not considered here.

Based on the flowchart for constructibility checks in **LRFD [Appendix C6]**, nominal yielding of both flanges must be checked as well as the flexural resistance of the compression flange. For discretely braced flanges (note f_l is taken as zero since this is an interior girder and there are no curvature, skew, deck overhang or wind load effects considered) **LRFD [6.10.3.2.1 & 6.10.3.2.2]**:

$$f_{bu} + f_l \leq \phi_f \cdot R_h \cdot F_{yf}$$

The flange stress, f_{bu}, is taken from Table E24-1.6-1 for the noncomposite dead load for the top flange since no deck placement analysis was performed. By inspection, since lateral flange bending is not considered, and no live load effects are considered, Strength IV is the controlling limit state and the compression flange is the controlling flange.

$f_{bu} := 1.5 \cdot 16.26$ ksi $\boxed{f_{bu} = 24.39}$ ksi

$\boxed{\phi_f \cdot R_h \cdot F_{yf} = 50.00}$ ksi OK

The flexural resistance calculation ensures that the compression flange has sufficient strength with respect to lateral torsional and flange local buckling based limit states, including the consideration of flange lateral bending where these effects are judged to be significant. The equation is in **LRFD [6.10.3.2]**:

$$f_{bu} + \frac{1}{3} \cdot f_l \leq \phi_f \cdot F_{nc}$$

Where:

F_{nc} = Nominal flexural resistance of the flange (ksi)

For straight I-girder bridges with compact or noncompact webs, the nominal resistance may be calculated from **LRFD [Appendix A6.3.3]** which includes the beneficial contribution of the St. Venant constant, J, in the calculation of the lateral torsional buckling resistance. This example will not use **LRFD [Appendix A6.3.3]**, but a check of the noncompact slenderness limit of web using **LRFD [6.10.6.2.3]** is included for reference.

$D_c := 28.73 - 0.75$ $\boxed{D_c = 27.98}$ in

$\boxed{\frac{2 \cdot D_c}{t_w} = 111.92}$

$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E_s}{F_{yc}}}$ $\boxed{\lambda_{rw} = 137.27}$

$\frac{2 \cdot D_c}{t_w} < 5.7 \cdot \sqrt{\frac{E}{F_{yc}}}$ OK

Although the noncomposite section has a nonslender web according to equation 1 of **LRFD [6.10.6.2.3]**, for this example, these beneficial effects will conservatively not be utilized.

The nominal flexural resistance of the compression flange is therefore taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance calculated according to **LRFD [6.10.8.2]**.

Local buckling resistance **LRFD [6.10.8.2.2]**:

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}}$$

Where:

λ_f = Slenderness ratio for the compression flange

b_{fc} = Full width of the compression flange (in)

t_{fc} = Thickness of the compression flange (in)

$b_{fc} := 14$ in (see Figure E24-1.2-1)

$t_{fc} := 0.75$ in (see Figure E24-1.2-1)

$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}}$ $\boxed{\lambda_f = 9.33}$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_{yc}}}$$

Where:

λ_{pf} = Limiting slenderness ratio for a compact flange

$\boxed{\lambda_{pf} = 9.15}$

Since $\lambda_f > \lambda_{pf}$, F_{nc} must be calculated by the following equation:

$$F_{nc} := \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

Where:

F_{yr} = Compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression-flange lateral bending, taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$

λ_{rf} = Limiting slenderness ratio for a noncompact flange

R_b = Web load-shedding factor **LRFD [6.10.1.10.2]**

$$F_{yr} := \max\left(\min\left(0.7 \cdot F_{yc}, F_{yw}\right), 0.5 \cdot F_{yc}\right)$$ $\boxed{F_{yr} = 35.00}$ ksi

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_s}{F_{yr}}}$$ $\boxed{\lambda_{rf} = 16.12}$

$R_b := 1.0$

$$F_{nc} := \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$\boxed{F_{nc} = 49.61}$ ksi

Lateral torsional buckling resistance **LRFD [6.10.8.2.3]**:

For the noncomposite loads during construction:

$Depth_{comp} := 55.625 - 26.897$ (see Figure E24-1.2-1 and Table E24-1.3-1)

$\boxed{Depth_{comp} = 28.73}$ in

The effective radius of gyration, r_t, for lateral torsional buckling is calculated as follows:

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}}$$

Where:

D_c = Depth of the web in compression in the elastic range (in). For composite sections see **LRFD [Appendix D6.3.1]**

$t_{topfl} := 0.75$ in

$D_c := Depth_{comp} - t_{topfl}$ $\boxed{D_c = 27.98}$ in

$$r_t := \frac{b_{fc}}{\sqrt{12\cdot\left(1+\frac{1}{3}\cdot\frac{D_c\cdot t_w}{b_{fc}\cdot t_{fc}}\right)}}$$ $\boxed{r_t = 3.36}$ in

The limiting unbraced length, L_p, to achieve the nominal flexural resistance of $R_bR_hF_{yc}$ under uniform bending is calculated as follows:

$$L_p := 1.0\cdot r_t\cdot\sqrt{\frac{E_s}{F_{yc}}}$$ $\boxed{L_p = 80.99}$ in

The limiting unbraced length, L_r, to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression-flange residual stress effects is calculated as follows:

$$L_r := \pi\cdot r_t\cdot\sqrt{\frac{E_s}{F_{yr}}}$$ $\boxed{L_r = 304.13}$ in

$L_b = 240.00$ in

The moment gradient correction factor, C_b, is computed as follows:

Note since f_{mid} is greater than f_2 at the location of maximum positive moment (see Figure E24-1.1-3), use C_b = 1.0 according to **LRFD [6.10.8.2.3]**.

$C_b := 1.00$

Therefore:

$$F_{nc} := C_b\cdot\left[1-\left(1-\frac{F_{yr}}{R_h\cdot F_{yc}}\right)\cdot\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]\cdot R_b\cdot R_h\cdot F_{yc}$$

$\boxed{F_{nc} = 39.3}$ ksi

Use

(minimum of local buckling and lateral torsional buckling) $F_{nc} := 39.3$ ksi

$\boxed{\phi_f\cdot F_{nc} = 39.30}$ ksi

$\boxed{f_{bu}+\frac{1}{3}\cdot(0) = 24.39}$ ksi OK

Web bend-buckling during construction must also be checked according to equation 3 of **LRFD [6.10.3.2.1]**. However, since the noncomposite section has previously been shown to have a nonslender web, web bend-buckling need not be checked in this case according to **LRFD [6.10.3.2.1]**.

In addition to checking the nominal flexural resistance during construction, the nominal shear resistance must also be checked **LRFD [6.10.3.2.3]**. However, shear is minimal at the location of maximum positive moment, and it is maximum at the pier in this case.

Therefore, for this design example, the nominal shear resistance for constructibility will be presented later for the girder design section at the pier.

E24-1.16 - Check Wind Effects on Girder Flanges - Positive Moment Region

As stated in previously, for this design example, the interior girder is being designed.

Wind effects generally do not control a steel girder design, and they are generally considered for the exterior girders only **LRFD [6.10.1.6 & C4.6.2.7.1]**. However, for this design example, wind effects will be presented later for the girder design section at the pier for illustration only.

Specification checks have been completed for the location of maximum positive moment, which is at 0.4L in Span 1.

E24-1.17 Check Section Proportion Limits - Negative Moment Region

Now the specification checks are repeated for the location of maximum negative moment, which is at the pier, as shown in Figure E24-1.17-1. This is also the location of maximum shear in this case.

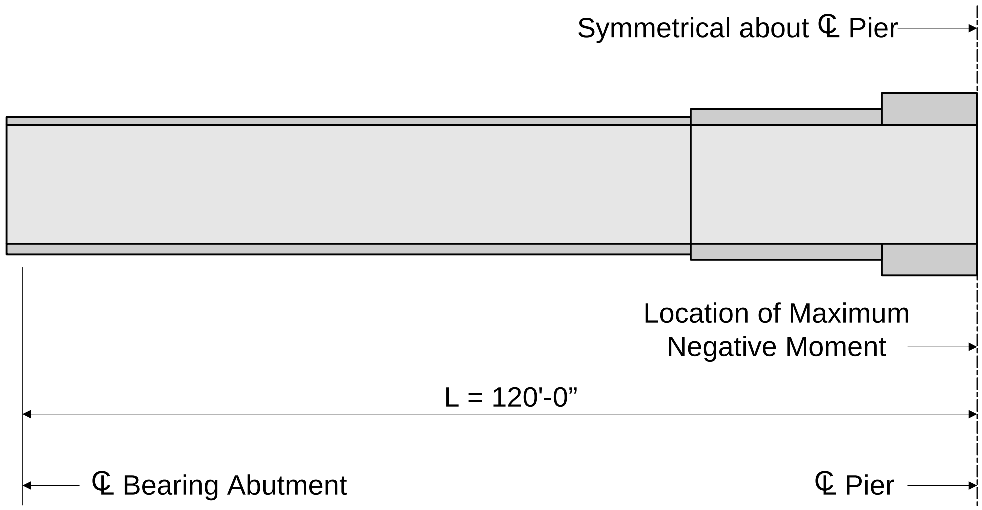


Figure E24-1.17-1
Location of Maximum Negative Moment

Several checks are required to ensure that the proportions of the girder section are within specified limits **LRFD [6.10.2]**.

The first section proportion check relates to the web slenderness **LRFD [6.10.2.1]**. For a section without longitudinal stiffeners, the web must be proportioned such that:

$$\frac{D}{t_w} \leq 150 \qquad \boxed{\frac{D}{t_w} = 108.00} \qquad \text{OK}$$

The second set of section proportion checks relate to the general proportions of the section

LRFD [6.10.2.2]. The compression and tension flanges must be proportioned such that:

$$\frac{b_f}{2 \cdot t_f} \le 12.0$$

$b_f := 14$

$t_f := 2.50$

$\boxed{\frac{b_f}{2 \cdot t_f} = 2.80}$ OK

$b_f \ge \frac{D}{6}$ $\boxed{\frac{D}{6} = 9.00}$ in OK

$t_f \ge 1.1 \cdot t_w$ $\boxed{1.1t_w = 0.55}$ in OK

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10$$

$I_{yc} := \frac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in^4

$I_{yt} := \frac{2.50 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in^4

$\boxed{\frac{I_{yc}}{I_{yt}} = 1.100}$ OK

E24-1.18 Compute Plastic Moment Capacity - Negative Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**. For composite sections in negative flexure, the concrete deck is ignored and the longitudinal deck reinforcement is included in the computation of M_p.

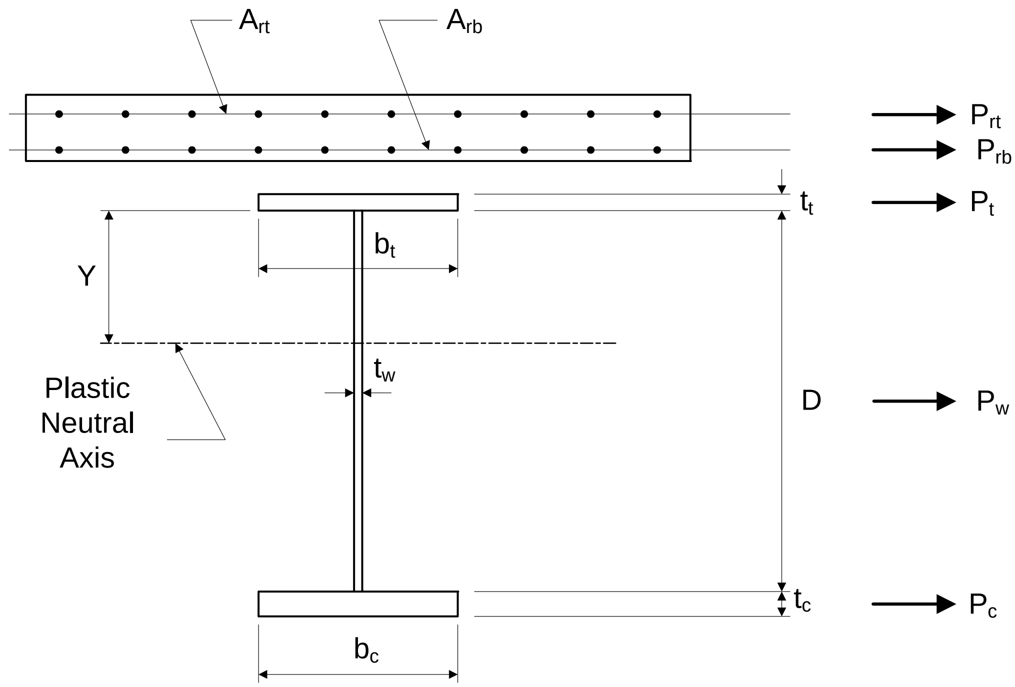


Figure E24-1.18-1
Computation of Plastic Moment Capacity for Negative Bending Sections

The plastic force in the tension flange, P_t, is calculated as follows:

$t_t := 2.50$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 1750}$ kips

The plastic force in the web, P_w, is calculated as follows:

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

The plastic force in the compression flange, P_c, is calculated as follows:

$t_c := 2.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 1925}$ kips

The plastic force in the top layer of longitudinal deck reinforcement, P_{rt}, used to compute the plastic moment is calculated as follows:

$P_{rt} := F_{yrt} \cdot A_{rt}$

Where:

F_{yrt} = Specified minimum yield strength of the top layer of longitudinal concrete deck reinforcement (ksi)

A_{rt} = Area of the top layer of longitudinal reinforcement within the effective concrete deck width (in^2)

$F_{yrt} := 60$ ksi

$$A_{rt} := 0.44 \cdot \left(\frac{W_{effflange} \cdot 12}{7.5} \right)$$ $\boxed{A_{rt} = 7.04}$ in^2

$$P_{rt} := F_{yrt} \cdot A_{rt}$$ $\boxed{P_{rt} = 422}$ kips

This example conservatively ignores the contribution from the bottom layer of longitudinal deck reinforcement, but the calculation is included for reference. The plastic force in the bottom layer of longitudinal deck reinforcement, P_{rb}, used to compute the plastic moment is calculated as follows:

$$P_{rb} := F_{yrb} \cdot A_{rb}$$

Where:

F_{yrb} = Specified minimum yield strength of the bottom layer of longitudinal concrete deck reinforcement (ksi)

A_{rb} = Area of the bottom layer of longitudinal reinforcement within the effective concrete deck width (in^2)

$F_{yrb} := 60$ ksi

$$A_{rb} := 0 \cdot \left(\frac{W_{effflange} \cdot 12}{1} \right)$$ $\boxed{A_{rb} = 0.00}$ in^2

$$P_{rb} := F_{yrb} \cdot A_{rb}$$ $\boxed{P_{rb} = 0}$ kips

Check the location of the plastic neutral axis, as follows:

$\boxed{P_c + P_w = 3275}$ kips

$\boxed{P_t + P_{rb} + P_{rt} = 2172}$ kips

$\boxed{P_c + P_w + P_t = 5025}$ kips

$\boxed{P_{rb} + P_{rt} = 422}$ kips

Therefore the plastic neutral axis is located within the web **LRFD [Appendix Table D6.1-2]**.

$$Y := \left(\frac{D}{2} \right) \cdot \left(\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1 \right)$$ $\boxed{Y = 22.05}$ in

Although it will be shown in the next design step that this section qualifies as a nonslender web section at the strength limit state, the optional provisions of Appendix A to **LRFD [6]** are not employed in this example. Thus, the plastic moment is not used to compute the flexural resistance and therefore does not need to be computed.

E24-1.19 Determine if Section is a Compact-Web, Noncompact-Web, or Slender-Web Section - Negative Moment Region

Since the section is in a straight bridge, the next step in the design process is to determine if the section is a compact-web, noncompact-web, or slender-web section. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

Where the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the noncompact-web slenderness limit, as follows **LRFD [6.10.6.2.3]**:

$$\frac{2 \cdot D_c}{t_w} \le 5.7 \sqrt{\frac{E_s}{F_{yc}}} \qquad \lambda_{rw} := 5.7 \sqrt{\frac{E_s}{F_{yc}}}$$

At sections in negative flexure, D_c of the composite section consisting of the steel section plus the longitudinal reinforcement is to be used at the strength limit state.

$D_c := 31.077 - 2.75$ (see Figure E24-1.2-1 and Table E24-1.3-3)

$\boxed{D_c = 28.33}$ in

$\boxed{\frac{2 \cdot D_c}{t_w} = 113.3}$

$\boxed{5.7 \cdot \sqrt{\frac{E_s}{F_{yc}}} = 137.3}$

The section is a nonslender web section (i.e. either a compact-web or noncompact-web section). Next, check:

$I_{yc} := \frac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in^4

$I_{yt} := \frac{2.5 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in^4

$\boxed{\frac{I_{yc}}{I_{yt}} = 1.10}$ > 0.3 OK

Therefore, the web qualifies to use the optional provisions of **LRFD [Appendix A6]** to compute the flexural resistance. However, since the web slenderness is closer to the noncompact web slenderness limit than the compact web slenderness limit in this case, the simpler equations of **LRFD [6.10.8]**, which assume slender-web behavior and limit the resistance to Fyc or below, will conservatively be applied in this example to compute the flexural resistance at the strength limit state. The investigation proceeds by calculating the flexural resistance of the discretely braced compression flange.

E24-1.20 Design for Flexure - Strength Limit State - Negative Moment Region

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance **LRFD [6.10.8.2.2 & 6.10.8.2.3]**.

Local buckling resistance **LRFD [6.10.8.2.2]**:

$b_{fc} := 14$ (see Figure E24-1.2-1)

$t_{fc} := 2.75$ (see Figure E24-1.2-1)

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}} \qquad \boxed{\lambda_f = 2.55}$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_{yc}}} \qquad \boxed{\lambda_{pf} = 9.15}$$

Since $\lambda_f < \lambda_{pf}$, F_{nc} is calculated using the following equation:

$$F_{nc} := R_b \cdot R_h \cdot F_{yc}$$

Since $2D_c/t_w$ is less than λ_{rw} (calculated above), R_b is taken as 1.0 **LRFD [6.10.1.10.2]**.

$\boxed{F_{nc} = 50.00}$ ksi

Lateral torsional buckling resistance **LRFD [6.10.8.2.3]**:

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}} \qquad \boxed{r_t = 3.81} \text{ in}$$

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E_s}{F_{yc}}} \qquad \boxed{L_p = 91.86} \text{ in}$$

$$L_r := \pi \cdot r_t \cdot \sqrt{\frac{E_s}{F_{yr}}} \qquad \boxed{L_r = 344.93} \text{ in}$$

$L_b = 240.00$

The moment gradient correction factor, C_b, is computed as follows:

Where the variation in the moment along the entire length between brace points is concave in shape, which is the case here, $f_1 = f_0$. (calculated below based on the definition of f_0 given in **LRFD [6.10.8.2.3]**).

$M_{NCDC0.8L} := 112.1 + 780.3 + 17.8$ $\boxed{M_{NCDC0.8L} = 910.20}$ kip-ft

$S_{NCDC0.8L} := 2278.2$ in^3

$M_{par0.8L} := 110.3$ kip-ft

The section properties specified for the 0.8 pt are the properties found at the pier based on **LRFD [6.10.8.2.3]**.

$M_{fws0.8L} := 104.4$ kip-ft

$M_{LL0.8L} := 1165.9$ kip-ft

$S_{rebar0.8L} := 2380.2$ in^3

$$f_1 := 1.25 \cdot \frac{M_{NCDC0.8L} \cdot 12}{S_{NCDC0.8L}} + 1.25 \cdot \frac{M_{par0.8L} \cdot 12}{S_{rebar0.8L}} + 1.50 \cdot \frac{M_{fws0.8L} \cdot 12}{S_{rebar0.8L}} + 1.75 \cdot \frac{M_{LL0.8L} \cdot 12}{S_{rebar0.8L}}$$

$\boxed{f_1 = 17.76}$ ksi

$f_2 := 48.84$ ksi (Table E24-1.6-2)

$\boxed{\frac{f_1}{f_2} = 0.36}$

$$C_b := 1.75 - 1.05 \cdot \left(\frac{f_1}{f_2}\right) + 0.3 \cdot \left(\frac{f_1}{f_2}\right)^2 \quad < 2.3$$

$\boxed{C_b = 1.41}$

Therefore:

$F_{yr} := \max\left(\min\left(0.7 \cdot F_{yc}, F_{yw}\right), 0.5 \cdot F_{yc}\right)$ $\boxed{F_{yr} = 35.00}$ ksi

$$F_{nc} := C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$\boxed{F_{nc} = 58.03}$ ksi

$F_{nc} \le R_b \cdot R_h \cdot F_{yc}$ $\boxed{R_b \cdot R_h \cdot F_{yc} = 50.00}$ ksi

Use:

$F_{nc} := 50.00$ ksi

$\boxed{\phi_f \cdot F_{nc} = 50.00}$ ksi

$f_{bu} := 48.84$ ksi (Table E24-1.6-2)

Since there are no curvature or skew effects and wind is not considered under the Strength I load combination, f_l is taken equal to zero. Therefore:

$\boxed{f_{bu} + \frac{1}{3}\cdot(0) = 48.84}$ ksi OK

The investigation proceeds by calculating the flexural resistance of the continuously braced tension flange **LRFD [6.10.8.1.3 & 6.10.8.3]**.

$f_{bu} \le \phi_f \cdot R_h \cdot F_{yf}$ $\boxed{\phi_f \cdot R_h \cdot F_{yf} = 50.00}$ ksi

(Table E24-1.6-2) $f_{bu} := 47.52$ ksi OK

E24-1.21 - Design for Shear - Negative Moment Region

Shear must be checked at each section of the girder. For this design example, shear is maximum at the pier.

The first step in the design for shear is to check if the web must be stiffened. The nominal shear resistance, V_n, of unstiffened webs of hybrid and homogeneous girders is **LRFD [6.10.9.2]**:

$$V_n := C \cdot V_p$$

Where:

C = Ratio of the shear-buckling resistance to the shear yield strength in accordance with **LRFD [6.10.9.3.2]**, with the shear-buckling coefficient, k, taken equal to 5.0

V_p = Plastic shear force (kips)

$k := 5.0$ $\boxed{\frac{D}{t_w} = 108.00}$

$$\boxed{1.12 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 60.31}$$

$$\boxed{1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 75.39}$$

Therefore,

$$\frac{D}{t_w} \ge 1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}}$$

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E_s \cdot k}{F_{yw}}\right) \qquad \boxed{C = 0.390}$$

The plastic shear force, V_p, is then:

$$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w \qquad \boxed{V_p = 783.0} \text{ kips}$$

$V_n := C \cdot V_p$ $\boxed{V_n = 305.6}$ kips

The factored shear resistance, V_r, is computed as follows **LRFD [6.10.9.1]**:

$\phi_v := 1.00$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 305.6}$ kips

The shear resistance at this design section is checked as follows:

$\Sigma\eta_i \cdot \gamma_i \cdot Q_i \le R_r$

Or in this case:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i \le V_r$

$\eta_i := 1.00$

As computed in E24-1.6, the factored Strength I Limit State shear at the pier is as follows:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i := 414.3$ kips

$\boxed{V_r = 305.6}$ kips

Since the shear resistance of an unstiffened web is less than the actual design shear, the web must be stiffened.

The transverse intermediate stiffener spacing is 120 inches. The spacing of the transverse intermediate stiffeners does not exceed 3D, therefore the design section can be considered stiffened and the provisions of **LRFD [6.10.9.3]** apply.

The section must be checked against the web to flange proportion limits for interior web panels **LRFD [6.10.9.3.2]**.

$$\frac{2 \cdot D \cdot t_w}{b_{fc} \cdot t_{fc} + b_{ft} \cdot t_{ft}} \le 2.5$$

Where:

b_{ft} = Full width of tension flange (in)

t_{ft} = Thickness of tension flange (in)

$b_{ft} := 14.0$

$t_{ft} := 2.50$

$$\boxed{\frac{2 \cdot D \cdot t_w}{b_{fc} \cdot t_{fc} + b_{ft} \cdot t_{ft}} = 0.73}$$ OK

The nominal shear resistance , V_n, of the interior web panel at the pier is then:

$$V_n := \left[V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \right] \right]$$

Where:

C = Ratio of the shear-buckling resistance to the shear yield strength

d_0 = Transverse stiffener spacing (in)

$d_o := 120$

$$k := 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2}$$ $\boxed{k = 6.01}$

$\boxed{\frac{D}{t_w} = 108.00}$

$\boxed{1.12 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 66.14}$

$\boxed{1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 82.67}$

$$\frac{D}{t_w} \geq 1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}}$$

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E_s \cdot k}{F_{yw}}\right)$$ $\boxed{C = 0.469}$

$V_p = 783.00$

$$V_n := V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \right]$$ $\boxed{V_n = 515.86}$ kips

The factored shear resistance, V_r, is computed as follows:

$\phi_v := 1.00$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 515.86}$ kips

As previously computed, for this design example:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i := 414.3$ kips

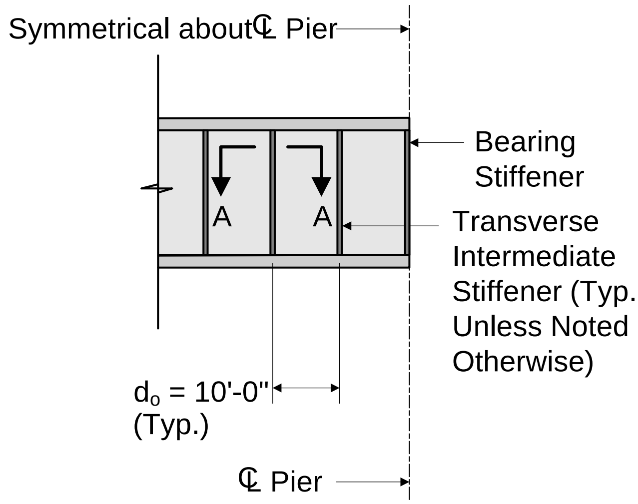
$\boxed{V_r = 515.86}$ kips OK

Therefore, the girder design section at the pier satisfies the shear resistance requirements for the web.

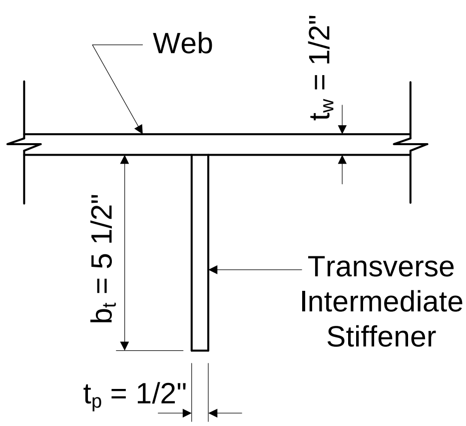
E24-1.22 Design Transverse Intermediate Stiffeners - Negative Moment Region

It is assumed that the transverse intermediate stiffeners consist of plates welded to one side of the web. The required interface between the transverse intermediate stiffeners and the top and bottom flanges is described in **LRFD [6.10.11.1.1]**.

The transverse intermediate stiffener configuration is assumed to be as presented in the following figure.



Partial Girder Elevation at Pier



Section A-A

Figure E24-1.22-1
Transverse Intermediate Stiffener

The first specification check is for the projecting width of the transverse intermediate stiffener. The width, b_t, of each projecting stiffener element must satisfy the following **LRFD [6.10.11.1.2]**:

$b_t \geq 2.0 + \frac{D}{30.0}$ and $16.0 \cdot t_p \geq b_t \geq 0.25 b_f$

Where:

t_p = Thickness of the projecting stiffener element (in)

b_f = Full width of the widest compression flange within the field section under consideration (in)

$b_t := 5.5$ in

$D := 54$ in

$t_p := 0.50$ in

$b_f = 14.00$ in

$\boxed{2.0 + \frac{D}{30.0} = 3.80}$ in OK

$\boxed{16.0 \cdot t_p = 8.00}$ in

$\boxed{0.25 \cdot b_f = 3.50}$ in OK

The moment of inertia, I_t, of the transverse stiffener must satisfy the following since each panel adjacent to the stiffener supports a shear force larger than the shear buckling resistance ($V_{cr} = CV_p$) **LRFD [6.10.11.1.3]**:

If $I_{t2} > I_{t1}$, then :

$$I_t \geq I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right)$$

Otherwise:

$$I_t \geq I_{t2}$$

$$I_{t1} := b \cdot t_w^3 \cdot J$$

Where:

b = The smaller of do and D (in)

J = Stiffener bending rigidity parameter

$b := \min\left(d_o, D\right)$ $b = 54.00$ in

$$J := \max\left[\frac{2.5}{\left(\frac{d_o}{D}\right)^2} - 2.0, 0.5\right] \qquad J = 0.50$$

$$I_{t1} := b \cdot t_w^{\,3} \cdot J = 3.38 \qquad \text{in}^4$$

$$I_{t2} := \frac{D^4 \cdot \rho_t^{\,1.3}}{40} \cdot \left(\frac{F_{yw}}{E}\right)^{1.5}$$

Where:

ρ_t = The larger of F_{yw}/F_{crs} and 1.0

The local buckling stress for the stiffener, F_{crs}, is calculated as follows:

$$F_{crs} := \frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2} \le F_{ys}$$

Where:

F_{ys} = Specified minimum yield strength of the stiffener (ksi)

$$\boxed{\frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2} = 74.30} \qquad \text{ksi}$$

$$F_{ys} := 50.00 \qquad \text{ksi}$$

Use

$$F_{crs} := \min\left[F_{ys}, \frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2}\right] \qquad \boxed{F_{crs} = 50.00} \qquad \text{ksi}$$

$$\rho_t := \max\left(\frac{F_{yw}}{F_{crs}}, 1.0\right) \qquad \boxed{\rho_t = 1.00}$$

$$\boxed{I_{t2} := \frac{D^4 \cdot \rho_t^{\,1.3}}{40} \cdot \left(\frac{F_{yw}}{E_s}\right)^{1.5} = 15.22} \qquad \text{in}^4$$

Since $I_{t2} > I_{t1}$, the moment of inertia, I_t, of the transverse stiffener must satisfy:

$$I_t \geq I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right)$$

$V_u := 414.3$ kip

$V_{cr} := C \cdot V_p = 367.53$ kip

$V_n = 515.86$ kip

$I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right) = 7.11$ in^4

$I_t := \frac{t_p \cdot b_t^3}{3}$ $\boxed{I_t = 27.73}$ in^4

Therefore,

$I_t \geq I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right)$ OK

E24-1.23 Design for Flexure - Fatigue and Fracture Limit State - Negative Moment Region

For this design example, sample nominal fatigue resistance computations were presented previously (E24-1.13) for the girder section at the location of maximum positive moment **LRFD [6.6.1]**. Detail categories are explained and illustrated in **LRFD [Table 6.6.1.2.3-1]**.

In addition to the nominal fatigue resistance computations, a special fatigue requirement for webs must also be checked **LRFD [6.10.5.3]**. This check is required to control out-of-plane flexing of the web due to shear under repeated live loading.

The check is made using fatigue range live load shear in combination with the shear due to the unfactored permanent load. This total shear is limited to the shear buckling resistance ($V_{cr} = CV_p$), as follows:

$V_u \leq V_{cr}$

Based on the unfactored shear values in Table E24-1.6-3:

$V_u = V_{noncomp} + V_{par} + V_{fws} + 1.5V_{LLfatiguerange}$

$V_u := 111.5 + 14.5 + 13.8 + (1.5 \cdot 47.4)$ $\boxed{V_u = 210.90}$ kips

$C = 0.469$ See E24-1.21

$V_p = 783.00$ kips See E24-1.21

$V_{cr} := C \cdot V_p$ $\boxed{V_{cr} = 367.53}$ kips

$V_u \le V_{cr}$ OK

Therefore, the special fatigue requirement for webs for shear is satisfied.

Other fatigue resistance calculations in the negative moment region are not shown here, but would be similar to the sample check illustrated previously for the positive moment region (E24-1.13).

E24-1.24 Design for Flexure - Service Limit State - Negative Moment Region

The girder must be checked for service limit state control of permanent deflection **LRFD [6.10.4]**. Service II Limit State is used for this check.

The flange stress checks of **LRFD [6.10.4.2.2]** will not control for composite sections in negative flexure for which the nominal flexural resistance under the strength load combinations given in **LRFD [Table 3.4.1-1]** is determined according to the slender-web provision of **LRFD [6.10.8]**, which is the case in this example.

However, for sections in negative flexure, the web must satisfy the web bend buckling check given by equation 4 of **LRFD [6.10.4.2.2]** at the service limit state, using the appropriate value of the depth of the web in compression in the elastic range, D_c.

$$f_c \le F_{crw}$$

$$F_{crw} := \frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2} \qquad \text{(LRFD 6.10.1.9.1-1)}$$

Where:

k = Bend-buckling coefficient = $9/(D_c/D)^2$

The factored Service II flexural stress was previously computed in Table E24-1.6-2 as follows:

$f_{botgdr} := -34.22$ ksi

$f_{topgdr} := 22.39$ ksi

As previously explained, for this design example, the concrete slab is assumed to be fully effective for both positive and negative flexure for service limit states. Therefore, when this assumption is made, D_c must be computed as follows as indicated in **LRFD [Appendix D6.3.1]**:

$$D_c := \left(\frac{-f_c}{|f_c| + f_t}\right) \cdot d - t_{fc} \ge 0$$

$Depth_{gdr} := 59.25$ in (see Figure E24-1.2-1)

$$Depth_{comp} := \frac{-f_{botgdr}}{|f_{botgdr}| + f_{topgdr}} \cdot Depth_{gdr} \qquad \boxed{Depth_{comp} = 35.82} \text{ in}$$

$t_{botfl} := 2.75$ in

$D_c := Depth_{comp} - t_{botfl}$ $\boxed{D_c = 33.07}$ in

$D := 54.0$ in

$$k := \frac{9.0}{\left(\frac{D_c}{D}\right)^2}$$ $\boxed{k = 24.00}$

$$\boxed{\frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2} = 53.71}$$ ksi

$$F_{crw} := \min\left[\frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2}, R_h \cdot F_{yc}, \frac{F_{yw}}{0.7}\right]$$ $\boxed{F_{crw} = 50.00}$ ksi

$t_{bf} := 2.75$ in

$$f_c := f_{botgdr} \cdot \left(\frac{D_c}{D_c + t_{bf}}\right)$$ $\boxed{f_c = -31.59}$ ksi OK

E24-1.25 Design for Flexure - Constructibility Check - Negative Moment Region

The girder must also be checked for flexure during construction **LRFD [6.10.3.2]**. The girder has already been checked in its final condition when it behaves as a composite section. The constructibility must also be checked for the girder prior to the hardening of the concrete deck when the girder behaves as a noncomposite section.

For discretely braced flanges in compression with a compact or noncompact web and with f_l equal to zero (interior girder), equation 2 is used. This check is similar to the check performed in E24-1.20 and will not be checked here.

For the interior girder in this case (where $f_l = 0$), the sizes of the flanges at the pier section are controlled by the strength limit state flexural resistance checks illustrated previously. Therefore, separate constructibility checks on the flanges need not be made. However, the web bend buckling resistance of the noncomposite pier section during construction must be checked according to equation 3 of **LRFD [6.10.3.2.1]**, as follows:

$$f_{bu} \le \phi_f \cdot F_{crw}$$

Check first if the noncomposite section at the pier is a nonslender web section. From Table E24-1.3-3 **LRFD [6.10.6.2.3]**:

$D_c := 28.718 - 2.75$ $\boxed{D_c = 25.97}$ in

$$\boxed{\frac{2 \cdot D_c}{t_w} = 103.87}$$

$$\boxed{\lambda_{rw} = 137.27}$$

$$\frac{2 \cdot D_c}{t_w} < \lambda_{rw} \qquad \text{OK}$$

The section is therefore a nonslender web section (i.e. a noncompact web section), web bend buckling need not be checked in this case according to **LRFD [6.10.3.2.1]**.

In addition to checking the flexural resistance during construction, the shear resistance in the web must also be checked prevent shear buckling of the web during construction as follows **LRFD [6.10.3.3]**:

$$V_{cr} := C \cdot V_p \qquad \boxed{V_{cr} = 367.53} \quad \text{kips}$$

$$V_r := \phi_v \cdot V_{cr} \qquad \boxed{V_r = 367.53} \quad \text{kips}$$

$$V_u := (1.25 \cdot 111.5) \qquad \boxed{V_u = 139.38} \quad \text{kips} \quad \text{OK}$$

Therefore, the design section at the pier satisfies the constructibility specification checks.

E24-1.26 Check Wind Effects on Girder Flanges - Negative Moment Region

Wind effects generally do not control a steel girder design, and they are generally considered for the exterior girders only **LRFD [C6.10.1.6 & C4.6.2.7.1]**. However, for illustrative purposes, wind effects are presented below for the girder design section at the pier. A bridge height of greater than 30 feet is used in this design step to illustrate the required computations **LRFD [3.8.1.1]**.

The stresses in the bottom flange are combined as follows **LRFD [6.10.8.1.1]**:

$$\left(f_{bu} + \frac{1}{3} f_l\right) \le \phi_f \cdot F_{nc}$$

$$f_l := \frac{6 \cdot M_w}{t_{fb} \cdot b_{fb}^2} \qquad \text{(LRFD 6.10.1.6)}$$

Since the deck provides horizontal diaphragm action and since there is wind bracing in the superstructure, the maximum wind moment, M_w, on the loaded flange is determined as follows:

$$M_w := \frac{W \cdot L_b^2}{10}$$

$$\frac{L_b}{12} = 20.00 \qquad \text{ft}$$

$$W := \frac{\eta \cdot \gamma \cdot P_D \cdot d}{2}$$

$\eta := 1.0$

$\gamma := 0.40$ for Strength V Limit State

Assume that the bridge is to be constructed in a city. The design horizontal wind pressure, P_D, is computed as follows **LRFD [3.8.1.2]**:

$$P_D := P_B \cdot \left(\frac{V_{DZ}}{V_B}\right)^2$$

Where:

P_B = Base wind pressure **LRFD [Table 3.8.1.1-1]** (ksf)

V_{DZ} = Design wind velocity at design elevation Z (mph)

V_B = Base wind velocity of 100 mph for a 30.0 ft height

$P_B := 0.050$ ksf

$V_B := 100$ mph

$$V_{DZ} := 2.5 \cdot V_o \cdot \left(\frac{V_{30}}{V_B}\right) \cdot \ln\left(\frac{Z}{Z_o}\right)$$

Where:

V_{30} = Wind velocity at 30.0 feet above low ground or above design water level (mph)

V_0 = Friction velocity **LRFD [Table 3.8.1.1-1]** (mph)

Z = Height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 30.0 feet

Z_0 = Friction length of upstream fetch **LRFD [Table 3.8.1.1-1]** (ft)

$V_o := 12.0$ MPH for a bridge located in a city

$V_{30} := 60$ MPH assumed wind velocity at 30 feet above low ground or above design water level at bridge site

$V_B = 100$ MPH

$Z := 35$ ft assumed height of structure at which wind loads are being calculated as measured from low ground or from water level

$Z_o := 8.20$ ft for a bridge located in a city

$$V_{DZ} := 2.5 \cdot V_o \cdot \left(\frac{V_{30}}{V_B}\right) \cdot \ln\left(\frac{Z}{Z_o}\right) \qquad \boxed{V_{DZ} = 26.12} \quad \text{MPH}$$

$$P_D := P_B \cdot \left(\frac{V_{DZ}}{V_B}\right)^2 \qquad \boxed{P_D = 0.0034} \quad \text{ksf}$$

$d := 8.45$ ft from bottom of girder to top of barrier

$$W := P_D \cdot d \qquad \boxed{W = 0.0288} \quad \text{kips/ft}$$

LRFD [3.8.1.2.1] states that the total wind loading, W, must not be taken less than 0.30 klf on beam or girder spans, therefore use P_D as computed below:

$W := 0.30$ kips/ft

$$P_D := \frac{W}{d} \qquad \boxed{P_D = 0.0355} \quad \text{ksf}$$

After the design horizontal wind pressure has been computed, the factored wind force per unit length applied to the flange is computed as follows **LRFD [C4.6.2.7.1]**:

$$W := \frac{\eta \cdot \gamma \cdot P_D \cdot d}{2} \qquad \boxed{W = 0.060} \quad \text{kips/ft}$$

Next, the maximum lateral moment in the flange due to the factored wind loading is computed as follows:

$$M_w := \frac{W \cdot \left(\frac{L_b}{12}\right)^2}{10} \qquad \boxed{M_w = 2.40} \quad \text{kip-ft}$$

Finally, the flexural stress at the edges of the bottom flange due to factored wind loading is computed as follows **LRFD [6.10.8.1.1]**:

$t_{fb} := 2.75$ in

$b_{fb} := 14.0$ in

$$f_l := \frac{6 \cdot -M_w \cdot 12}{t_{fb} \cdot b_{fb}^2} \qquad \boxed{f_l = -0.321} \quad \text{ksi}$$

The load factor for live load is 1.35 for the Strength V Limit State. However, it is 1.75 for the Strength I Limit State, which we have already investigated. Therefore, it is clear that wind effects will not control the design of this steel girder. Nevertheless, the following computations are presented simply to demonstrate that wind effects do not control this design:

$$f_{bu} := [(1.25 \cdot -16.56) + (1.25 \cdot -2.05)] + [(1.50 \cdot -1.94) + (1.35 \cdot -12.26)]$$

$\boxed{f_{bu} = -42.72}$ ksi

$\boxed{f_{bu} + \frac{1}{3} f_l = -42.83}$ ksi

$\boxed{F_{nc} = 50.00}$ ksi

$f_{bu} + \frac{1}{3} f_l \le \phi_f \cdot F_{nc}$ OK

E24-1.27 Draw Schematic of Final Steel Girder Design

Since all of the specification checks were satisfied, the trial girder section presented in E24-1.2 is acceptable. If any of the specification checks were not satisfied or if the design were found to be overly conservative, then the trial girder section would need to be revised appropriately, and the specification checks would need to be repeated for the new trial girder section.

The following is a schematic of the final steel girder configuration:

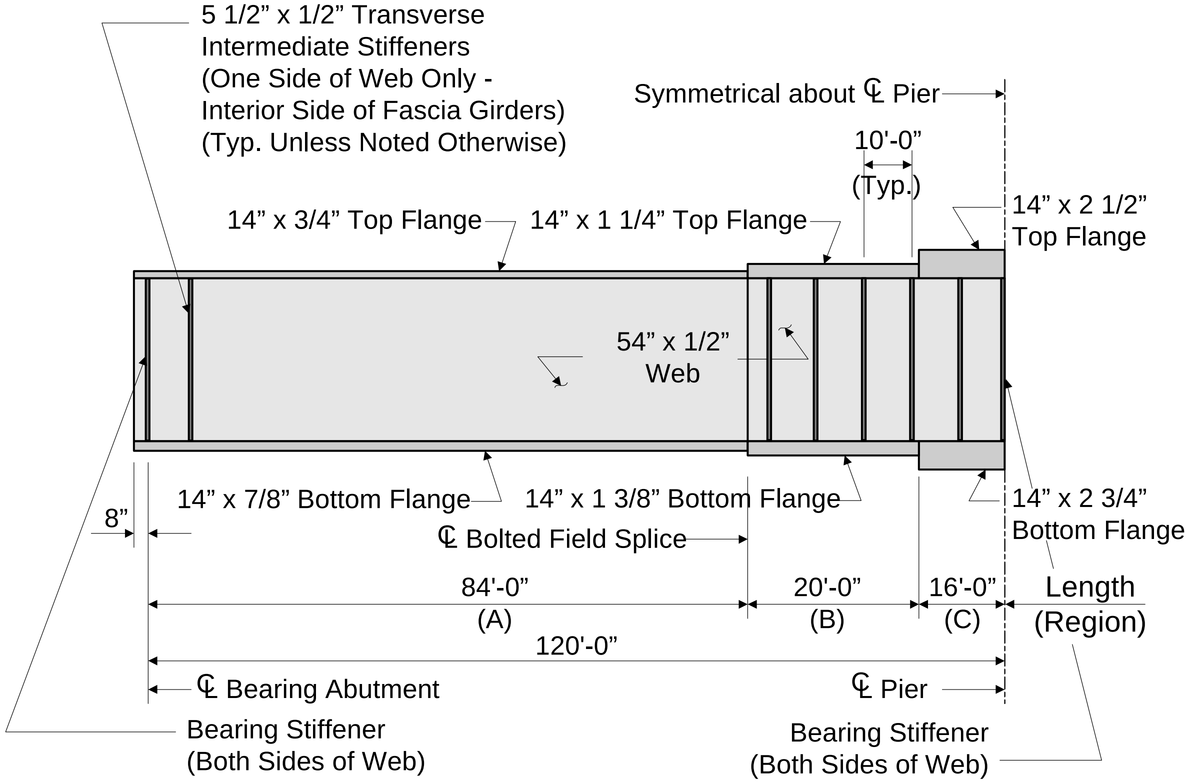


Figure E24-1.27-1
Final Plate Girder Elevation

For this design example, only the location of maximum positive moment, the location of maximum negative moment, and the location of maximum shear were investigated. However, the above schematic shows the plate sizes and stiffener spacing throughout the entire length of the girder.

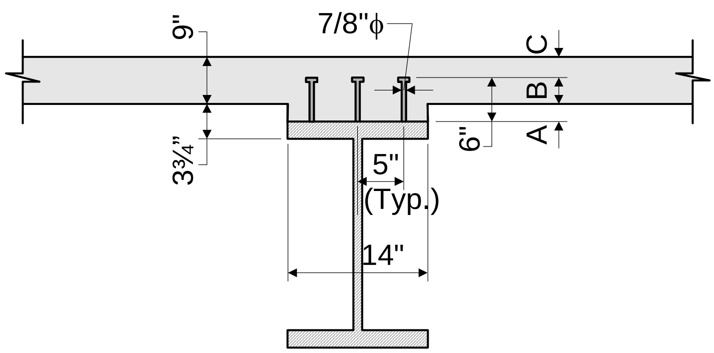
Design computations for shear connectors and bearing stiffeners now follow.

E24-1.28 Design Shear Connectors

For continuous composite bridges, shear connectors are normally provided throughout the length of the bridge. In the negative flexure region, since the longitudinal reinforcement is considered to be a part of the composite section, shear connectors must be provided **LRFD [6.10.10.1]**.

Studs are used as shear connectors. The shear connectors must permit a thorough compaction of the concrete to ensure that their entire surfaces are in contact with the concrete. In addition, the shear connectors must be capable of resisting both horizontal and vertical movement between the concrete and the steel.

The following figure shows the stud shear connector proportions, as well as the location of the stud head within the concrete deck.



<u>Figure E24-1.28-1</u>
Stud Shear Connectors

Shear Connector Embedment			
Flexure Region	**A**	**B**	**C**
Positive	3.00"	3.00"	6.00"
Intermediate	2.50"	3.50"	5.50"
Negative	1.25"	4.75"	4.25"

<u>Table E24-1.28-1</u>
Shear Connector Embedment

The ratio of the height to the diameter of a stud shear connector must not be less than 4.0 **LRFD [6.10.10.1.1]**. For this design example, the ratio is computed based on the dimensions presented in Figure E24-1.28-1, as follows:

$Height_{stud} := 6.0$ in

$Diameter_{stud} := 0.875$ in

$$\frac{Height_{stud}}{Diameter_{stud}} = 6.86$$ OK

The pitch of the shear connectors must be determined to satisfy the fatigue limit state as specified in **LRFD [6.10.10.2 & 6.10.10.3]**, as applicable. The resulting number of shear connectors must not be less than the number required to satisfy the strength limit states as specified in **LRFD [6.10.10.4]**.

The pitch, p, of the shear connectors must satisfy the following equation **LRFD [6.10.10.1.2]**:

$$p \le \frac{n \cdot Z_r}{V_{sr}}$$

Where:

n = Number of shear connectors in a cross-section

Z_r = Shear fatigue resistance of an individual shear connector **LRFD [6.10.10.2]** (kip)

V_{sr} = Horizontal fatigue shear range per unit length (kip-in)

The shear fatigue resistance of an individual shear connector, Z_r, is taken as:

$ADTT_{SL} = 3000$ > 960, Therefore, use Fatigue 1 load combinations with fatigue shear resistance for infinite life as follows:

$$Z_r := 5.5 \cdot d^2$$

Where:

d = Diameter of the stud (in)

The horizontal fatigue shear range per unit length, V_{sr}, is taken as:

$$V_{sr} := \sqrt{V_{fat}^2 + F_{fat}^2}$$

Where:

V_{fat} = Longitudinal fatigue shear range per unit length

F_{fat} = Radial fatigue shear range per unit length (kip-in)

The longitudinal fatigue shear range per unit length, V_{fat}, is taken as:

$$V_{fat} := \frac{V_f \cdot Q}{I}$$

Where:

V_f = Vertical shear force range under the fatigue load combination in **LRFD [Table 3.4.1-1]** with the fatigue live load taken as specified in **LRFD [3.6.1.4]** (kip)

Q = First moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section (in^3)

I = Moment of inertia of the short-term composite section (in^4)

The radial fatigue shear range per unit length, F_{fat}, is taken as the larger of:

$$F_{fat1} := \frac{A_{bot} \cdot \sigma_{flg} \cdot l}{w \cdot R}$$

$$F_{fat2} := \frac{F_{rc}}{w}$$

Where:

A_{bot} = Area of the bottom flange (in^2)

σ_{flg} = Range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending (ksi)

l = Distance between brace points (ft)

w = Effective length of deck (in) taken as 48.0 in, except at end supports where w may be taken as 24.0 in

R = Minimum girder radius within the panel (ft)

F_{rc} = Net range of cross-frame or diaphragm force at the top flange (kip)

Since this bridge utilizes straight spans and has no skew, the radial fatigue shear range, F_{fat} is taken as zero. Therefore:

$$V_{sr} := V_{fat}$$

In the positive flexure region, the maximum fatigue live load shear range is located at the abutment. For illustration purposes, this example uses the average fatigue live load shear range in the positive moment region and assumes it acts at 0.4L. In reality, the required pitch should be calculated throughout the entire length of the girder. The actual pitch should be chosen such that it is less than or equal to the required pitch. The factored average value is computed as follows:

$V_f := 1.5 \cdot (44.5)$ $\boxed{V_f = 66.75}$ kips

The parameters I and Q are based on the short-term composite section and are determined using the deck within the effective flange width. In the positive flexure region:

$n := 3$ (see Figure E24-1.28-1)

$I := 70696.16$ in^4 (see Table E24-1.3-1)

$Q := \left[\frac{(8.5) \cdot (120)}{8}\right] \cdot (62.875 - 52.777)$ $\boxed{Q = 1287.49}$ in^3

$V_{fat} := \frac{V_f \cdot Q}{I}$ $\boxed{V_{fat} = 1.22}$ kip/in

$V_{sr} := V_{fat}$ $\boxed{V_{sr} = 1.22}$ kip/in

$d := 0.875$ in

$Z_r := 5.5 \cdot d^2$ $\boxed{Z_r = 4.21}$ kips

$p := \dfrac{n \cdot Z_r}{V_{sr}}$ $\boxed{p = 10.39}$ in

In the negative flexure region:

$n := 3$ (see Figure E24-1.28-1)

From **LRFD [C6.10.10.1.2]**, in the negative flexure region, the parameters I and Q may be determined using the reinforcement within the effective flange width for negative moment, unless the concrete slab is considered to be fully effective for negative moment in computing the longitudinal range of stress, as permitted in **LRFD [6.6.1.2.1]**. For this design example, I and Q are assumed to be computed considering the concrete slab to be fully effective.

$I := 139158.7$ in^4 (see Table E24-1.3-3)

$Q := \left[\dfrac{(8.5) \cdot (120)}{8}\right] \cdot (64.750 - 48.868)$ $\boxed{Q = 2024.95}$ in^3

$V_f := 1.5 \cdot (47.4)$ $\boxed{V_f = 71.10}$ kips

$V_{fat} := \dfrac{V_f \cdot Q}{I}$ $\boxed{V_{fat} = 1.03}$ kip/in

$V_{sr} := V_{fat}$ $\boxed{V_{sr} = 1.03}$ kip/in

$p := \dfrac{n \cdot Z_r}{V_{sr}}$ $\boxed{p = 12.21}$ in

Therefore, based on the above pitch computations to satisfy the fatigue limit state, use the following pitch throughout the entire girder length:

$p := 10$ in

As stated earlier, the shear connector pitch typically is not the same throughout the entire length of the girder. In reality, most girder designs use a variable pitch, which is beneficial economically.

However, for simplicity in this design example, a constant shear connector pitch of 10 inches will be used.

In addition, the shear connectors must satisfy the following pitch requirements **LRFD [6.10.10.1.2]**:

$p \leq 24$ in OK

$p \geq 6 \cdot d$ $\boxed{6 \cdot d = 5.25}$ in OK

For transverse spacing, the shear connectors must be placed transversely across the top flange of the steel section and may be spaced at regular or variable intervals **LRFD [6.10.10.1.3]**.

Stud shear connectors must not be closer than 4.0 stud diameters center-to-center transverse to the longitudinal axis of the supporting member.

$4 \cdot d = 3.50$ in

$Spacing_{transverse} := 5.0$ in (see Figure E24-1.28-1) OK

In addition, the clear distance between the edge of the top flange and the edge of the nearest shear connector must not be less than 1.0 inch.

$D_{clear} := \frac{14}{2} - 5 - \frac{d}{2}$ $\boxed{D_{clear} = 1.56}$ in OK

The clear depth of concrete cover over the tops of the shear connectors should not be less than 2.0 inches, and shear connectors should penetrate at least 2.0 inches into the deck **LRFD [6.10.10.1.4]**. Based on the shear connector penetration information presented in Table E24-1.28-1, both of these requirements are satisfied.

For the strength limit state, the factored resistance of the shear connectors, Q_r, is computed as follows **LRFD [6.10.10.4.1]**:

$Q_r := \phi_{sc} \cdot Q_n$

$\phi_{sc} := 0.85$ (LRFD 6.5.4.2)

The nominal shear resistance of one stud shear connector embedded in a concrete slab, Q_n, is computed as follows **LRFD [6.10.10.4.3]**:

$Q_n := 0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c} \leq A_{sc} \cdot F_u$

Where:

A_{sc} = Cross-sectional area of a stud shear connector (in^2)

F_u = Specified minimum tensile strength of a stud shear connector from **LRFD [6.4.4]** (ksi)

$A_{sc} := \pi \cdot \frac{d^2}{4}$ $\boxed{A_{sc} = 0.601}$ in^2

$F_u := 60.0$ ksi

$E_c := 3834$ ksi

$Q_n := \min\left(0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c}, A_{sc} \cdot F_u\right)$ $\boxed{Q_n = 36.08}$ kips

$Q_r := \phi_{sc} \cdot Q_n$ $\boxed{Q_r = 30.67}$ kips

The number of shear connectors provided over the section being investigated must not be less than the following **LRFD [6.10.10.4.1]**:

$$n := \frac{P}{Q_r}$$

For continuous spans that are composite for negative flexure in their final condition, the nominal shear force, P, must be calculated for the following regions **LRFD [6.10.10.4.2]**:

1. Between points of maximum positive design live load plus impact moments and adjacent ends of the member

2. Between points of maximum positive design live load plus impact moment and centerlines of adjacent interior supports

For Region 1:

$$P := \sqrt{P_p^{\,2} + F_p^{\,2}}$$

Where:

P_p = Total longitudinal shear force in the concrete deck at the point of maximum positive live load plus impact moment (kips)

F_p = Total radial shear force in the concrete deck at the point of maximum positive live load plus impact moment (kips)

The total longitudinal shear force in the concrete deck at the point of maximum positive live load plus impact moment, P_p, is taken as the lesser of:

$$P_{1p} := 0.85 \cdot f'_c \cdot b_s \cdot t_s$$

or

$$P_{2p} := F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}$$

$t_{ft} := 0.875$ in (see E24-1.27)

$t_{fc} := 0.75$ in (see E24-1.27)

$$P_p := \min\left(0.85 \cdot f'_c \cdot b_s \cdot t_s, F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}\right)$$

$\boxed{P_p = 2488}$ kips

For straight spans or segments, F_p may be taken equal to zero which gives **LRFD [6.10.10.4.2]**:

$P := P_p$ $\boxed{P = 2488}$ kips

Therefore, the number of shear connectors provided between the section of maximum positive moment and each adjacent end of the member must not be less than the following **LRFD [6.10.10.4.1]**:

$n := \dfrac{P}{Q_r}$ $\boxed{n = 81.1}$

For region 2:

$$P := \sqrt{P_T^{\,2} + F_T^{\,2}}$$

Where:

P_T = Total longitudinal shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kips)

F_T = Total radial shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kips)

The total longitudinal shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support, P_T, is taken as:

$$P_T := P_p + P_n$$

Where:

P_n = Total longitudinal shear force in the concrete deck over an interior support (kips)

The total longitudinal shear force in the concrete deck over an interior support, P_n, is taken as the lesser of:

$$P_{1n} := F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}$$

or

$$P_{2n} := 0.45 \cdot f'_c \cdot b_s \cdot t_s$$

$t_{ft} := 2.5$ in (see E24-1.27)

$t_{fc} := 2.75$ in (see E24-1.27)

$$P_n := \min\left(F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}, 0.45 \cdot f'_c \cdot b_s \cdot t_s\right)$$

$\boxed{P_n = 1836}$ kips

$P_T := P_p + P_n$ $\boxed{P_T = 4324}$ kips

For straight spans or segments, F_T may be taken equal to zero which gives:

$P := P_T$ $\boxed{P = 4324}$ kips

Therefore, the number of shear connectors provided between the section of maximum positive moment and the centerline of the adjacent interior pier must not be less than the following **LRFD [6.10.10.4.1]**:

$$n := \frac{P}{Q_r} \qquad \boxed{n = 141.0}$$

The distance between the end of the girder and the location of maximum positive moment is approximately equal to:

$L := 48.0$ ft (see Table E24-1.4-2)

Using a pitch of 10 inches, as previously computed for the fatigue limit state, and using the above length, the number of shear connectors provided is as follows:

$$n := 3 \cdot \frac{L \cdot (12)}{p} \qquad \boxed{n = 172.8} \quad \text{OK}$$

Similarly the distance between the section of the maximum positive moment and the interior support is equal to:

$L := 120.0 - 48.0$ $\boxed{L = 72.0}$ ft (see Table E24-1.4-2)

Using a pitch of 10 inches, as previously computed for the fatigue limit state, and using the above length, the number of shear connectors provided is as follows:

$$n := 3 \cdot \frac{L \cdot (12)}{p} \qquad \boxed{n = 259.2} \quad \text{OK}$$

Therefore, using a pitch of 10 inches for each row, with three stud shear connectors per row, throughout the entire length of the girder satisfies both the fatigue limit state requirements of **LRFD [6.10.10.1.2 & 6.10.10.2]** and the strength limit state requirements of **LRFD [6.10.10.4]**.

Use a shear stud spacing as illustrated in the following figure.

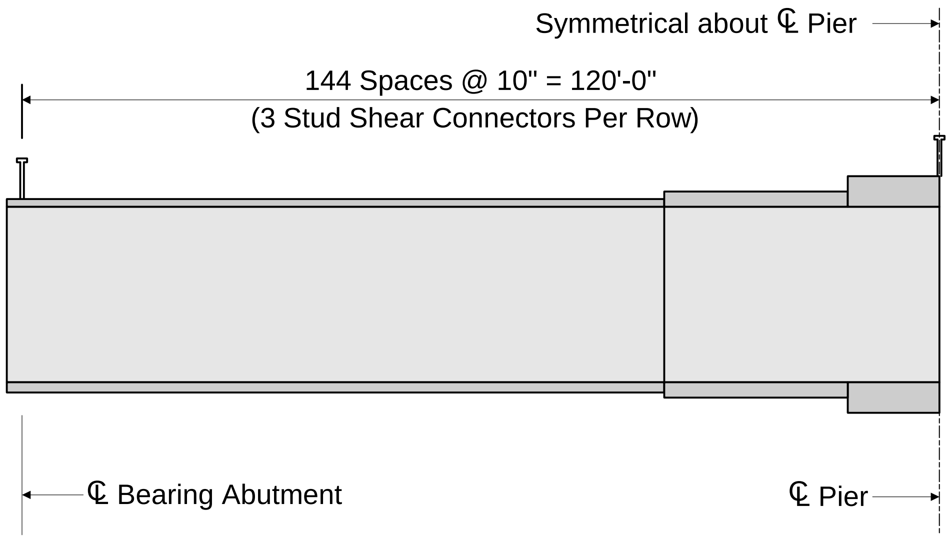


Figure E24-1.28-2
Shear Connector Spacing

E24-1.29 Design Bearing Stiffeners

Bearing stiffeners are required to resist the bearing reactions and other concentrated loads, either in the final state or during construction **LRFD [6.10.11.2.1]**.

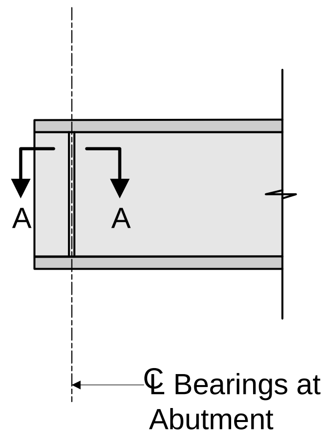
For plate girders, bearing stiffeners are required to be placed on the webs at all bearing locations. At all locations supporting concentrated loads where the loads are not transmitted through a deck or deck system, either bearing stiffeners are to be provided or the web must satisfy the provisions of **LRFD [Appendix D6.5]**.

Therefore, for this design example, bearing stiffeners are required at both abutments and at the pier. The following design of the abutment bearing stiffeners illustrates the bearing stiffener design procedure.

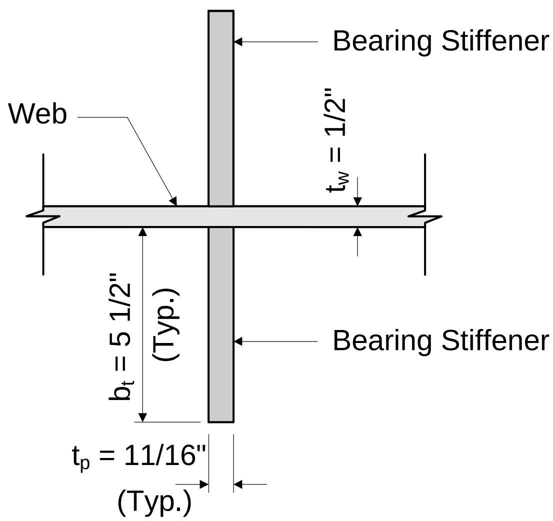
The bearing stiffeners in this design example consist of one plate welded to each side of the web. The connections to the web will be designed to transmit the full bearing force due to factored loads and is presented in E24-1.30.

The stiffeners extend the full depth of the web and, as closely as practical, to the outer edges of the flanges.

The following figure illustrates the bearing stiffener layout at the abutments.



Partial Girder Elevation at Abutment



Section A-A

Figure E24-1.29-1
Bearing Stiffeners at Abutments

The projecting width, b_t, of each bearing stiffener element must satisfy the following equation **LRFD [6.10.11.2.2]**. This provision is intended to prevent local buckling of the bearing stiffener plates.

$$b_t \leq 0.48 \cdot t_p \cdot \sqrt{\frac{E}{F_{ys}}}$$

Where:

t_p = Tickness of the projecting stiffener element (in)

F_{ys} = Specified minimum yield strength of the stiffener (ksi)

$b_t := 5.5$ in (see Figure E24-1.29-1)

$t_p := \frac{11}{16}$ in (see Figure E24-1.29-1)

$F_{ys} := 50$

$\boxed{0.48 \cdot t_p \cdot \sqrt{\frac{E_s}{F_{ys}}} = 7.95}$ in OK

The bearing resistance must be sufficient to resist the factored reaction acting on the bearing stiffeners **LRFD [6.10.11.2.3]**. The factored bearing resistance, R_{sbr}, is computed as follows:

$R_{sbr} := \phi_b \cdot R_{sbn}$

$\phi_b := 1.00$ (LRFD 6.5.4.2)

$R_{sbn} := 1.4 \cdot A_{pn} \cdot F_{ys}$

Where:

A_{pn} = Area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange (in^2)

Part of the stiffener must be clipped to clear the web-to-flange weld. Thus the area of direct bearing is less than the gross area of the stiffener. The bearing area, A_{pn}, is taken as the area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange. This is illustrated in the following figure:

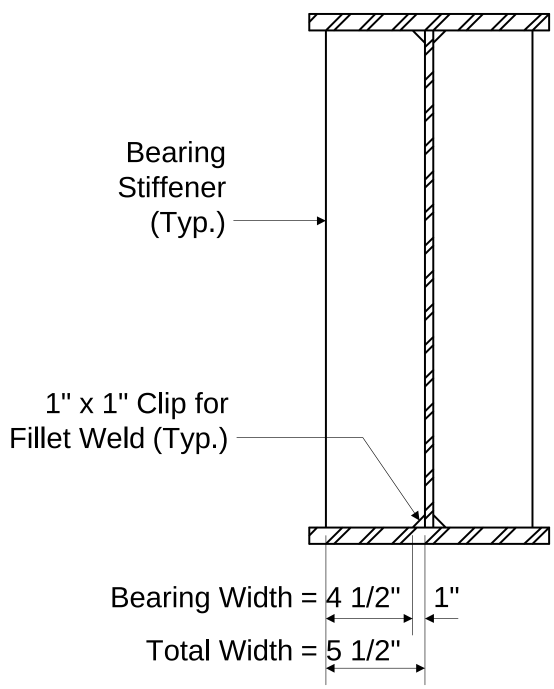


Figure E24-1.29-2
Bearing Width

$b_{brg} := b_t - 1.0$ $\boxed{b_{brg} = 4.50}$ in

$A_{pn} := 2b_{brg} \cdot t_p$ $\boxed{A_{pn} = 6.19}$ in^2

$R_{sbr} := \phi_b \cdot 1.4 \cdot A_{pn} \cdot F_{ys}$ $\boxed{R_{sbr} = 433.13}$ kips

The factored bearing reaction at the abutment is computed as follows, using load factors as presented in **LRFD [Table 3.4.1-1 & Table 3.4.1-2]** and using reactions obtained from Table E24-1.4-3 and Table E24-1.5-2:

$React_{Factored} := (1.25 \cdot 63.7) + (1.50 \cdot 7.4) + (1.75 \cdot 112.8)$

$\boxed{React_{Factored} = 288.13}$ kips

Therefore, the bearing stiffener at the abutment satisfies the bearing resistance requirements.

The final bearing stiffener check relates to the axial resistance of the bearing stiffeners **LRFD [6.10.11.2.4]**. The factored axial resistance is determined as specified in **LRFD [6.9.2.1]**. The radius of gyration is computed about the midthickness of the web, and the effective length is taken as 0.75D, where D is the web depth **LRFD [6.10.11.2.4a]**.

For stiffeners consisting of two plates welded to the web, the effective column section consists of the two stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the stiffeners **LRFD [6.10.11.2.4.b]**. This is illustrated in the following figure:

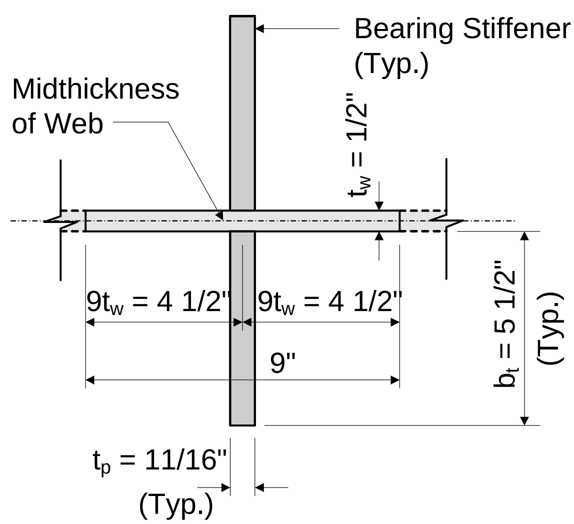


Figure E24-1.29-3
Bearing Stiffener Effective Section

$P_r := \phi_c \cdot P_n$ (LRFD 6.9.2.1)

$\phi_c := 0.90$ (LRFD 6.5.4.2)

Bearing stiffeners only need to be designed for Flexural Buckling failure (Torsional Buckling and Flexural Torsional Buckling are not applicable) **LRFD [6.9.4.1.1]**.

First, calculate the elastic critical buckling resistance, Pe, based on **LRFD [6.9.4.1.2]**.

$$P_e := \frac{A_g \cdot \left(\pi^2 \cdot E_s\right)}{\left(\frac{kl}{r_s}\right)^2}$$

Where:

kl = Taken as 0.75D, where D is the web depth (in)

r_s = Radius of gyration about the midthickness of the web (in)

A_g = Cross-sectional area of the effective section (in^2)

$kl := (0.75) \cdot (54)$ $\boxed{kl = 40.50}$ in

$$I_s := \frac{\left(0.6875 \cdot 11.5^3\right) + \left(8.3125 \cdot 0.5^3\right)}{12}$$ $\boxed{I_s = 87.22}$ in^4

$A_g := (0.6875 \cdot 11.5) + (8.3125 \cdot 0.5)$ $\boxed{A_g = 12.06}$ in^2

$$r_s := \sqrt{\frac{I_s}{A_g}}$$ $\boxed{r_s = 2.69}$ in

$$P_e := \frac{A_g \cdot \left(\pi^2 \cdot E_s\right)}{\left(\frac{kl}{r_s}\right)^2}$$ $P_e = 15220$ kip

Next, calculate the equivalent nominal yield resistance, P_o, given as:

$P_o := Q \cdot F_y \cdot A_g$ (LRFD 6.9.4.1.1)

Where:

Q = slender element reduction factor, taken as 1.0 for bearing stiffeners

$P_o := 1.0 F_y \cdot A_g$ $P_o = 603$ kip

$$\frac{P_e}{P_o} = 25.23$$ Since $P_e/P_o > 0.44$, Use equation 1 from **LRFD [6.9.4.1.1]**.

$$P_n := \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o$$ $\boxed{P_n = 593.20}$ kips

$$P_r := \phi_c \cdot P_n$$ $\boxed{P_r = 533.88}$ kips

$\boxed{React_{Factored} = 288.13}$ kips OK

Therefore, the bearing stiffener at the abutment satisfies the axial bearing resistance requirements.

The bearing stiffener at the abutment satisfies all bearing stiffener requirements. Use the bearing stiffener as presented in Figure E24-1.29-2 and Figure E24-1.29-3.

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E24-2 Bolted Field Splice, LRFD

E24-2.1 Obtain Design Criteria

This splice design example shows design calculations conforming to the *AASHTO LRFD Bridge Design Specifications (Seventh Edition - 2015 Interims)* as supplemented by the *WisDOT Bridge Manual (January 2015)*.
Note: This example uses the girder from example E24-1.

Presented in Figure E24-2.1-1 is the steel girder configuration and the bolted field splice location. **LRFD [6.13.6.1.4a]** recommends locating splices near points of dead load contraflexure.

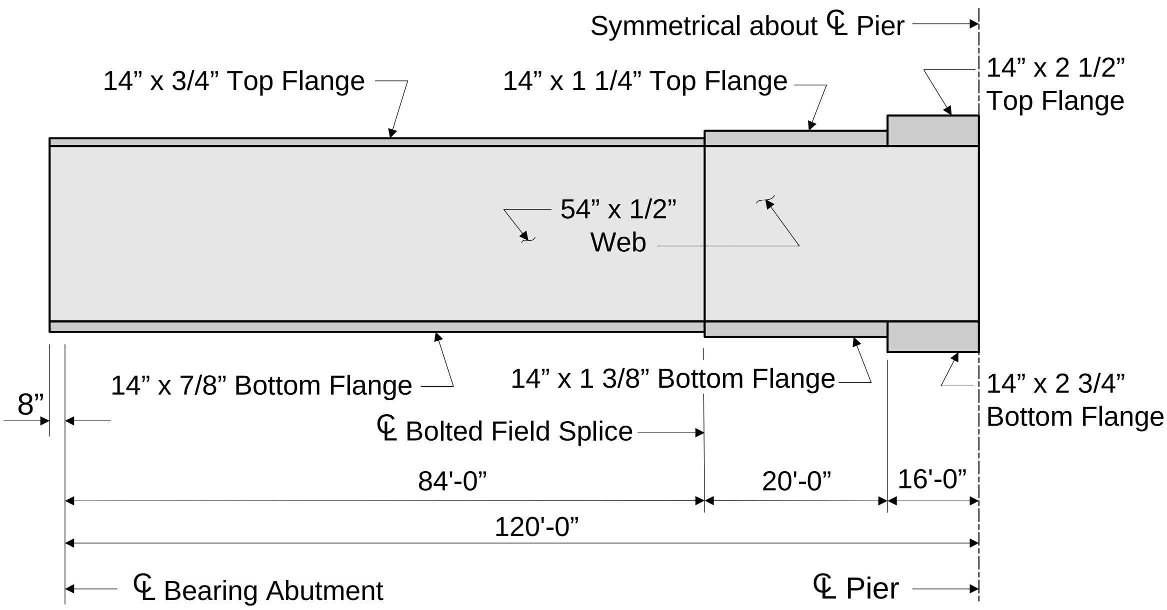

Figure E24-2.1-1
Plate Girder Elevation

The steel properties of the girder and splice plates are as follows:

Yield strength:

$F_y := 50$ ksi

Tensile strength:

$F_u := 65$ ksi

For specification checks requiring the flange yield strength:

$F_{yf} := 50$ ksi

The plate dimensions of the girder on the left side of the splice from Figure E24-2.1-1 are as follows:

Web thickness:

$t_w := 0.50$ in

Web depth:

$D := 54$ in

Top flange width:

$b_{fltL} := 14$ in

Top flange thickness:

$t_{fltL} := 0.75$ in

Bottom flange width:

$b_{flbL} := 14$ in

Bottom flange thickness:

$t_{flbL} := 0.875$ in

The plate dimensions of the girder on the right side of the splice from Figure E24-2.1-1 are as follows:

Web thickness:

$t_w := 0.50$ in

Web depth:

$D := 54$ in

Top flange width:

$b_{fltR} := 14$ in

Top flange thickness:

$t_{fltR} := 1.25$ in

Bottom flange width:

$b_{flbR} := 14$ in

Bottom flange thickness:

$t_{flbR} := 1.375$ in

The properties of the splice bolts are as follows:

Bolt diameter:

$d_{bolt} := 0.875$ in **LRFD [6.13.2.5]**

Bolt hole diameter (for design purposes add 1/16" to standard hole diameter):

$d_{hole} := 1.0$ in **LRFD [6.13.2.4.2-1]**

Bolt tensile strength:

$Fu_{bolt} := 120$ ksi **LRFD [6.4.3.1]**

The properties of the concrete deck are as follows:

Effective slab thickness:

$t_{seff} := 8.5$ in

Modular ratio:

$n := 8$

Haunch depth (measured from top of web):

$d_{haunch} := 3.75$ in

Effective flange width:

$W_{eff} := 120$ in

The area of longitudinal deck reinforcing steel in the negative moment region is for the top and bottom mat is given as number 6 bars at 7.5 inch spacing. The area of steel in the effective flange width is then:

For the top steel:

$$A_{deckreinftop} := (0.44) \cdot \frac{W_{eff}}{7.5} \qquad \boxed{A_{deckreinftop} = 7.04} \ in^2$$

For the bottom steel:

$$A_{deckreinfbot} := (0.44) \cdot \frac{W_{eff}}{7.5} \qquad \boxed{A_{deckreinfbot} = 7.04} \ in^2$$

Resistance factors **LRFD [6.5.4.2]**:

Flexure: $\phi_f := 1.0$

Shear: $\phi_v := 1.0$

Axial compression, composite: $\phi_c := 0.90$

Tension, fracture in net section: $\phi_u := 0.80$

Tension, yielding in gross section: $\phi_y := 0.95$

Bolts bearing on material: $\phi_{bb} := 0.80$

A325 and A490 bolts in shear: $\phi_s := 0.80$

Block shear: $\phi_{bs} := 0.80$

E24-2.2 Select Girder Section as Basis for Field Splice Design

Where a section changes at a splice, the smaller of the two connected sections shall be used in the design **LRFD [6.13.6.1.1]**. Therefore, the bolted field splice will be designed based on the left adjacent girder section properties. This will be referred to as the Left Girder throughout the calculations. The girder located to the right of the bolted field splice will be designated the Right Girder.

E24-2.3 Compute Flange Splice Design Loads

A summary of the unfactored moments at the splice from example 24-1 are listed below. The live loads include impact and distribution factors.

Dead load moments:

Noncomposite:

$M_{NDL} := -98.8$ kip-ft

Composite:

$M_{CDL} := 6.6$ kip-ft

Future wearing surface:

$M_{FWS} := 6.3$ kip-ft

Live load moments:

HL-93 positive:

$M_{PLL} := 1245.7$ kip-ft

HL-93 negative:

$M_{NLL} := -957.7$ kip-ft

Fatigue positive:

$M_{PFLL} := 375.6$ kip-ft

Fatigue negative:

$M_{NFLL} := -285.4$ kip-ft

Typically, splices are designed for the Strength I, Service II, and Fatigue I load combinations. The load factors for these load combinations are shown in Table E24.2.3-1 **LRFD [Tables 3.4.1-1 & 3.4.1-2]**:

	Load Factors					
	Strength I		Service II		Fatigue I	
Load	max	min	max	min	max	min
DC	1.25	0.90	1.00	1.00	-	-
DW	1.50	0.65	1.00	1.00	-	-
LL	1.75	1.75	1.30	1.30	1.50	1.50

Table E24-2.3-1
Load Factors

Flange stress computation procedure:

The stresses corresponding to the load combinations described above will be computed at the midthickness of the top and bottom flanges. The appropriate section properties and load factors for use in computing stresses are described below. Where necessary, refer to the signs of the previously documented design moments.

Strength I load combination:

The flexural stresses due to the factored loads at the strength limit state and for checking slip of the bolted connections at the point of splice shall be determined using the gross section properties **LRFD [6.13.6.1.4a]**.

Case 1: Dead load + positive live load

For this case, stresses will be computed using the gross section properties. The minimum load factor is used for the DC dead loads (noncomposite and composite) and the maximum load factor is used for the future wearing surface. The composite dead load and future wearing surface act on the 3n-composite slab section and the live load acts on the n-composite slab section **LRFD [6.10.1.1.1b]**.

Case 2: Dead load + negative live load

For this case, stresses will be computed using the gross section properties. The future wearing surface is excluded and the maximum load factor is used for the DC dead loads. The live load acts on the composite steel girder plus longitudinal reinforcement section. The composite dead load is applied to this section as well, as a conservative assumption for simplicity and convenience, since the net effect of the live load is to induce tension in the slab. **LRFD [6.10.1.1.1c]**.

Service II load combination:

Case 1: Dead load + positive live load

For this case, stresses will be computed using the gross steel section. The future wearing surface is included and acts, along with the composite dead load, on the 3n-composite slab section. The live load acts on the n-composite slab section.

Case 2: Dead load + negative live load

For this case, stresses will be computed using the gross steel section. The future wearing surface is excluded. The composite dead load acts on the 3n-composite slab section. The live load acts on the n-composite slab section.

Fatigue I load combination:

Case 1: Positive live load

For this case, stresses will be computed using the gross steel section. The live load acts on the n-composite slab section.

Case 2: Negative live load

For this case, stresses will be computed using the gross steel section. The live load acts on the n-composite slab section.

Section properties:

The effective flange area, A_e, is calculated from **LRFD [6.13.6.1.4c]**:

$$A_e := \left(\frac{\phi_u \cdot F_u}{\phi_y \cdot F_{yt}}\right) \cdot A_n \le A_g$$

Where:

ϕ_u	= Resistance factor for fracture of tension members **LRFD [6.5.4.2]**
ϕ_y	= Resistance factor for yielding of tension members **LRFD [6.5.4.2]**
A_n	= Net area of the tension flange (in^2) **LRFD [6.8.3]**
A_g	= Gross area of the tension flange (in^2)
F_u	= Specified minimum tensile strength of the tension flange (ksi) **LRFD [Table 6.4.1-1]**
F_{yt}	= Specified minimum yield strength of the tension flange (ksi)

The gross area of the top and bottom flange of the steel girder is as follows:

$A_{gbot} := t_{flbL} \cdot b_{flbL}$ $\boxed{A_{gbot} = 12.25}$ in^2

$A_{gtop} := t_{fltL} \cdot b_{fltL}$ $\boxed{A_{gtop} = 10.50}$ in^2

The net area of the bottom flange of the steel girder is defined as the product of the thickness of the flange and the smallest net width **LRFD [6.8.3]**. The net width is determined by subtracting from the width of the flange the sum of the widths of all holes in the assumed failure chain, and then adding the quantity s^2 /4g for each space between consective holes in the chain. Since the bolt holes in the flanges are lined up transverse to the loading direction, the governing failure chain is straight across the flange (i.e., s^2 /4g is equal to zero).

The net area of the bottom and top flanges of the steel girder now follows:

$A_{nbot} := \left(b_{flbL} - 4 \cdot d_{hole}\right) \cdot t_{flbL}$ $\boxed{A_{nbot} = 8.75}$ in^2

$A_{ntop} := \left(b_{fltL} - 4 \cdot d_{hole}\right) \cdot t_{fltL}$ $\boxed{A_{ntop} = 7.50}$ in^2

With the gross and net areas identified, the effective tension area of the bottom and top flanges can now be computed as follows:

$F_{yt} := 50$ ksi

$A_{ebot} := \left(\frac{\phi_u \cdot F_u}{\phi_y \cdot F_{yt}}\right) \cdot A_{nbot}$ $\boxed{A_{ebot} = 9.58}$ in^2

$A_{etop} := \left(\frac{\phi_u \cdot F_u}{\phi_y \cdot F_{yt}}\right) \cdot A_{ntop}$ $\boxed{A_{etop} = 8.21}$ in^2

Check:

$A_{ebot} = 9.58$ < $A_{gbot} = 12.25$ OK

Effective bottom flange area:

$A_{ebot} = 9.58$ in^2

Effective top flange area:

$A_{etop} = 8.21$ in^2

The transformed effective area of the concrete flange of the steel girder is now determined as follows:

$$A_c = \frac{\text{Effective Slab Width}}{\text{Modular Ratio}} \times t_{seff}$$

$W_{eff} = 120.00$ in

$n = 8$ in

$t_{seff} = 8.50$ in

For the n-composite beam:

$A_c := \frac{W_{eff}}{n} \cdot t_{seff}$ $\boxed{A_c = 127.50}$ in^2

For the 3n-composite beam:

$A_{c3n} := \frac{W_{eff}}{3n} \cdot t_{seff}$ $\boxed{A_{c3n} = 42.50}$ in^2

The section properties for the Left Girder are calculated with the aid of Figure E24-2.3-1 shown below:

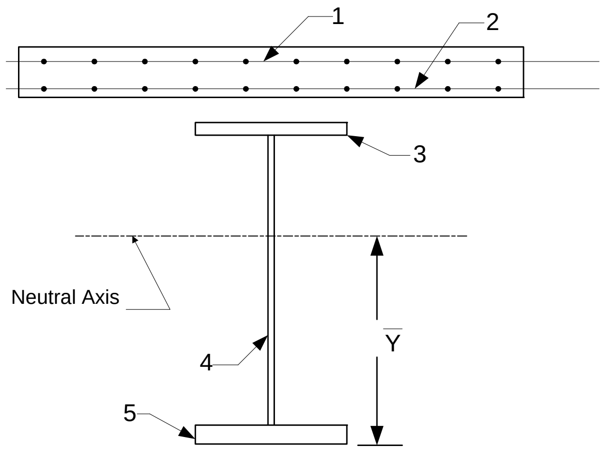

Figure E24-2.3-1
Girder, Slab and Longitudinal Reinforcement

The following tables contain the section properties for the left (i.e., smaller) girder section at the splice location. The properties in Table E24-2.3-2 are based on the gross area of the steel girder, and these properties are used for computation of stresses for the Service II and Fatigue Limit States. The properties in Tables E24-2.3-3 and E24-2.3-4 are based on the effective top flange and effective bottom flange of the steel girder, respectively, and these properties are used for computation of stresses for the Strength I Limit State.

Section Properties - Effective Top Flange Area						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	A*y^2 (Inches^4)	Itotal (Inches^4)
Girder only:						
Top flange	10.500	55.250	580.1	0.5	8441.1	8441.6
Web	27.000	27.875	752.6	6561.0	25.8	6586.8
Bottom flange	12.250	0.438	5.4	0.8	8576.1	8576.9
Total	49.750	26.897	1338.1	6562.3	17043.0	23605.3
Deck Steel:						
Girder	49.750	26.897	1338.1	23605.3	1066.7	24672.0
Top Steel	7.040	64.250	452.3	0.0	7538.3	7538.3
Total	56.790	31.527	1790.4	23605.3	8605.0	32210.3
Composite (3n):						
Girder	49.750	26.897	1338.1	23605.3	13668.5	37273.7
Slab	42.500	62.875	2672.2	255.9	16000.2	16256.0
Total	92.250	43.472	4010.3	23861.1	29668.6	53529.8
Composite (n):						
Girder	49.750	26.897	1338.1	23605.3	33321.4	56926.6
Slab	127.500	62.875	8016.6	767.7	13001.9	13769.5
Total	177.250	52.777	9354.7	24372.9	46323.2	70696.2
Section	y_{botmid} (Inches)	y_{topmid} (Inches)	S_{botweb} (Inches^3)	S_{botmid} (Inches^3)	S_{topmid} (Inches^3)	S_{topweb} (Inches^3)
Girder only	26.459	28.353	907.1	892.1	832.5	843.7
Deck Steel	31.090	23.723	1050.8	1036.0	1357.8	1379.6
Composite (3n)	43.035	11.778	1256.7	1243.9	4544.9	4694.4
Composite (n)	52.339	2.473	1362.1	1350.7	28583.9	33692.3

Table E24-2.3-2
Section Properties

Note: This example uses only the top layer of deck steel as it is WisDOT's policy to only include the top layer of bar steel in determining section properties and stresses.

Strength I Limit State stresses - Dead load + positive live load:

The section properties for this case have been calculated in Table E24-2.3-2. The stresses at the midthickness of the flanges are shown in Table E24-2.3-3, which immediately follows the sample calculation presented below.

A typical computation for the stresses occurring at the midthickness of the flanges is presented in the example below. The stress in the bottom flange of the girder is computed using the 3n-composite section for the composite dead load and future wearing surface, and the n-composite section for the live load:

$$f := \frac{M}{S}$$

Noncomposite DL:

Stress at the midthickness:

$f := f_{botgdr_1}$

Noncomposite DL Moment:

$M_{NDL} = -98.80$ kip-ft

Section modulus (girder only), from Table E24-2.3-2:

$S_{botgdr_1} := 892.1$ in^3

Stress due to the noncomposite dead load:

$$f_{botgdr_1} := \frac{M_{NDL} \cdot 12}{S_{botgdr_1}} \qquad \boxed{f_{botgdr_1} = -1.33} \text{ ksi}$$

Composite DL:

Stress at the midthickness:

$f := f_{botgdr_2}$

Composite DL moment:

$M_{CDL} = 6.60$ kip-ft

Section modulus (3n-composite), from Table E24-2.3-2:

$S_{botgdr_2} := 1243.9$ in^3

Stress due to the composite dead load:

$$f_{botgdr_2} := \frac{M_{CDL} \cdot 12}{S_{botgdr_2}} \qquad \boxed{f_{botgdr_2} = 0.06} \text{ ksi}$$

Future wearing surface:

Stress at the midthickness:

$f := f_{botgdr_3}$

FWS moment:

$M_{FWS} = 6.30$ kip-ft

Section modulus (3n-composite), From Table E24-2.3-2:

$S_{botgdr_3} := 1243.9$ in^3

Stress due to the composite dead load:

$$f_{botgdr_3} := \frac{M_{FWS} \cdot 12}{S_{botgdr_3}} \qquad \boxed{f_{botgdr_3} = 0.06} \text{ ksi}$$

Positive live load:

Stress at the midthickness:

$f := f_{botgdr_4}$

Live load moment:

$M_{PLL} = 1245.70$ kip-ft

Section modulus (n-composite), From Table E24-2.3-2:

$S_{botgdr_4} := 1350.7$ in^3

Stress due to the positive live load:

$$f_{botgdr_4} := \frac{M_{PLL} \cdot 12}{S_{botgdr_4}} \qquad \boxed{f_{botgdr_4} = 11.07} \text{ ksi}$$

The preceding stresses are now factored by their respective load factors to obtain the final factored stress at the midthickness of the bottom flange for this load case. The applicable load factors for this case were discussed previously **LRFD [Table 3.4.1-1 & 3.4.1-2]**.

$$f_{botgdr} := \left(0.90 \cdot f_{botgdr_1} + 0.90 \cdot f_{botgdr_2} + 1.50 \cdot f_{botgdr_3} + 1.75 \cdot f_{botgdr_4}\right)$$

$$\boxed{f_{botgdr} = 18.32} \text{ ksi}$$

The stresses at the midthickness of the top flange for this load case are computed in a similar manner. The section properties used to obtain the stresses in the top flange are also from Table E24-2.3-2.

The top and bottom flange midthickness stresses are summarized in Table E24-2.3-3, shown below.

Strength I - Dead Load + Positive Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.06	-0.02
FWS DL	6.30	0.06	-0.02
Live Load - HL-93	1245.70	11.07	-0.52
Summary of Factored Values			
Limit State			
Strength I	2106.45	18.32	0.33

Table E24-2.3-3
Strength I Flange Stresses for Dead + Pos. LL

The computation of the midthickness flange stresses for the remaining load cases are computed in a manner similar to what was shown in the sample calculation that preceded Table E24-2.3-3.

Strength I Limit State - Dead Load + Negative Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Strength I - Dead Load + Negative Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.08	-0.06
Live Load - HL-93	-957.70	-11.09	8.46
Summary of Factored Values			
Limit State			
Strength I	-1791.23	-20.98	16.52

Table E24-2.3-4
Strength I Flange Stresses for Dead + Neg. LL

Service II Limit State - Dead Load + Positive Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Service II - Dead Load + Positive Live Load			
Summary of Unfactored Values			
Loading	**Moment (K-ft)**	**f_{botmid} (ksi)**	**f_{topmid} (ksi)**
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.06	-0.02
FWS	6.30	0.06	-0.02
Live Load - HL-93	1245.70	11.07	-0.52
Summary of Factored Values			
Limit State			
Service II	1533.51	13.18	0.71

Table E24-2.3-5
Service II Flange Stresses for Dead + Pos. LL

Service II Limit State - Dead Load + Negative Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Service II - Dead Load + Negative Live Load			
Summary of Unfactored Values			
Loading	**Moment (K-ft)**	**f_{botmid} (ksi)**	**f_{topmid} (ksi)**
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.06	0.00
Live Load - HL-93	-957.70	-8.51	0.40
Summary of Factored Values			
Limit State			
Service II	-1337.21	-12.33	1.94

Table E24-2.3-6
Service II Flange Stresses for Dead + Neg. LL

Fatigue Limit State - Positive Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Fatigue - Positive Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Live Load-Fatigue	375.60	3.34	-0.16
Summary of Factored Values			
Limit State			
Fatigue	563.40	5.01	-0.24

Table E24-2.3-7
Fatigue Flange Stresses for Positive LL

Fatigue Limit State - Negative Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Fatigue - Negative Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Live Load-Fatigue	-285.40	-2.54	0.12
Summary of Factored Values			
Limit State			
Fatigue	-428.10	-3.80	0.18

Table E24-2.3-8
Fatigue Flange Stresses for Negative LL

Fatigue Limit State:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Fatigue - Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botweb} (ksi)	f_{topweb} (ksi)
Live Load-Pos	375.60	3.31	-0.13
Live Load-Neg	-285.40	-2.51	0.10
Summary of Factored Values			
Limit State			
Pos Fatigue	563.40	4.96	-0.20
Neg Fatigue	-428.10	-3.77	0.15

Table 42E2.3-9
Fatigue Web Stresses for Positive and Negative Live Load

A summary of the factored stresses at the midthickness of the top and bottom flanges for the Strength I, Service II, and Fatigue limit states are presented below in Tables E24-2.3-9 through E24-2.3-12. Table E24-2.3-12 also contains the top and bottom web fatigue stresses.

		Stress (ksi)	
Limit State	Location	Dead + Pos. LL	Dead + Neg. LL
Strength I	Bottom Flange	18.32	-20.98
	Top Flange	0.33	16.52

Table E24-2.3-10
Strength I Flange Stresses

		Stress (ksi)	
Limit State	Location	Dead + Pos. LL	Dead + Neg. LL
Service II	Bottom Flange	13.18	-12.33
	Top Flange	0.71	1.94

Table E24-2.3-11
Service II Flange Stresses

		Stress (ksi)	
Limit State	Location	Positive LL	Negative LL
Fatigue	Bottom Flange	5.01	-3.80
	Top Flange	-0.24	0.18
	Bottom of Web	4.96	-3.77
	Top of Web	-0.20	0.15

Table E24-2.3-12
Fatigue Flange and Web Stresses

Strength I minimum design force - controlling flange **LRFD [6.13.6.1.4c]**:

The next step is to determine the minimum design forces for the controlling flange of each load case (i.e., positive and negative live load). By inspection of Table E24-2.3-10, it is obvious that the bottom flange is the controlling flange for both positive and negative live load for the Strength I Limit State.

The minimum design force for the controlling flange, P_{cu}, is taken equal to the design stress, F_{cf}, times the smaller effective flange area, A_e, on either side of the splice. When a flange is in compression, the effective compression flange area shall be taken as $A_e = A_g$.

The calculation of the minimum design force is presented below for the load case of dead load with positive live load.

The minimum design stress for the controlling (bottom) flange, F_{cf}, is computed as follows **LRFD [6.13.6.1.4c]**:

$$F_{cf} := \frac{\left(\frac{f_{cf}}{R_h}\right) + \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g}{2} \geq 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g$$

Where:

f_{cf} = Maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the point of splice (ksi)

R_g = Flange resistance modification factor **LRFD [6.13.6.1.4c-3]**

R_h = Hybrid factor **LRFD [6.10.1.10.1]**. For hybrid sections in which F_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0

α = 1.0, except that a lower value equal to (F_n/F_{yf}) may be used for flanges where F_n is less than F_{yf}

ϕ_f = Resistance factor for flexure **LRFD [6.5.4.2]**

F_n = Nominal flexural resistance of the flange (ksi)

F_{yf} = Specified minimum yield strength of the flange (ksi)

Maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the point of splice (from Table E24-2.3-10):

$f_{cf} := 18.32$ ksi

$R_h := 1.0$

$R_g := 1.0$

$\alpha := 1.0$

Resistance factor for flexure (see E24-2.1.1):

$\phi_f = 1.0$

$F_{yf} = 50.00$ ksi

$$F_{cf_1} := \frac{\left(\frac{f_{cf}}{R_h}\right) + \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g}{2} \qquad \boxed{F_{cf_1} = 34.16} \text{ ksi}$$

Compute the minimum required design stress:

$F_{cf_2} := 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g$ $\boxed{F_{cf_2} = 37.50}$ ksi

The minimum design stress for the bottom flange for this load case is:

$F_{cf} := \max\left(F_{cf_1}, F_{cf_2}\right)$ $\boxed{F_{cf} = 37.50}$ ksi

The minimum design force now follows:

$P_{cu} := F_{cf} \cdot A_e$

The gross area of the bottom flange is:

$A_{flbL} := b_{flbL} \cdot t_{flbL}$ $\boxed{A_{flbL} = 12.25}$ in^2

Since the bottom flange force for this load case is a tensile force, the effective area will be used. This value was computed previously to be:

$A_{ebot} = 9.58$ in^2

Therefore:

$P_{cu} := F_{cf} \cdot A_{ebot}$ $\boxed{P_{cu} = 359.21}$ kips

Table E24-2.3-13 presents the minimum design forces for the Strength I Limit State for both the positive and negative live load cases.

		Strength I Limit State			
		Controlling Flange			
Load Case	**Location**	**f_{cf} (ksi)**	**F_{cf} (ksi)**	**Area (in^2)**	**P_{cu} (kips)**
Dead + Pos. LL	**Bot. Flange**	18.32	37.50	9.58	359.21
Dead + Neg. LL	**Bot. Flange**	-20.98	37.50	12.25	459.38

Table E24-2.3-13
Controlling Flange Forces

In the above table, the design controlling flange force (P_{cu}) is a compressive force for negative live load.

Strength I minimum design force - noncontrolling flange **LRFD [6.13.6.1.4c]**:

The next step is to determine the minimum design forces for the noncontrolling flange of each load case (i.e., positive and negative live load). By inspection of Table 24E2.3-10, the top flange is the noncontrolling flange for both positive and negative live load for the Strength I Limit State.

The minimum design force for the noncontrolling flange, P_{ncu}, is taken equal to the design stress, F_{ncf}, times the smaller effective flange area, A_e, on either side of the splice. When a flange is in compression, the effective compression flange area shall be taken as $A_e = A_g$

The calculation of the minimum design force is presented below for the load case of dead load with positive live load.

The minimum design stress for the noncontrolling (top) flange, F_{ncf}, is computed as follows **LRFD [6.13.6.1.4c]**:

$$F_{ncf} := R_{cf} \cdot \left| \frac{f_{ncf}}{R_h} \right| \geq 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g$$

Where:

R_{cf} = The absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange

f_{ncf} = Flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with f_{cf} (ksi)

Maximum flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with fcf (see Table 24E2.3-10):

$f_{ncf} := 0.33$ ksi

Controlling flange design stress:

$F_{cf} = 37.50$ ksi

Controlling flange actual stress:

$f_{cf} = 18.32$ ksi

Controlling flange stress ratio:

$$R_{cf} := \left| \frac{F_{cf}}{f_{cf}} \right| \qquad \boxed{R_{cf} = 2.05}$$

$R_h = 1.00$

Therefore:

$$F_{ncf_1} := R_{cf} \cdot \left| \frac{f_{ncf}}{R_h} \right| \qquad \boxed{F_{ncf_1} = 0.68} \text{ ksi}$$

Compute the minimum required design stress:

$$F_{ncf_2} := 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g \qquad \boxed{F_{ncf_2} = 37.50} \text{ ksi}$$

The minimum design stress in the top flange is:

$$F_{ncf} := \max\left(F_{ncf_1}, F_{ncf_2}\right) \qquad \boxed{F_{ncf} = 37.50} \text{ ksi}$$

The minimum design force now follows:

$$P_{ncu} := F_{ncf} \cdot A_e$$

For the positive live load case, the top flange is in compression. The effective compression flange area shall be taken as:

$A_{etop} := A_{gtop}$ $\boxed{A_{etop} = 10.50}$ in^2

Therefore:

$P_{ncu} := F_{ncf} \cdot A_{etop}$ $\boxed{P_{ncu} = 393.75}$ kips (compression)

Table E24-2.3-14 presents the minimum design forces for the Strength I Limit State for both the positive and negative live load cases.

		Strength I Limit State Noncontrolling Flange			
Load Case	**Location**	**f_{cf} (ksi)**	**F_{cf} (ksi)**	**Area (in^2)**	**P_{cu} (kips)**
Dead + Pos. LL	**Top Flange**	0.33	37.50	10.50	393.75
Dead + Neg. LL	**Top Flange**	16.52	37.50	8.21	307.89

Table E24-2.3-14
Noncontrolling Flange Forces

In the above table, the design noncontrolling flange force (P_{ncu}) is a compressive force for positive live load.

Service II Limit State flange forces **LRFD [6.13.6.1.4c]**:

Per the specifications, bolted connections for flange splices are to be designed as slip-critical connections for the service level flange design force. This design force shall be taken as the Service II design stress, F_s, multiplied by the smaller gross flange area on either side of the splice.

The Service II design stress, F_s, for the flange under consideration at a point of splice is defined as follows:

$$F_s := \frac{f_s}{R_h}$$

Where:

f_s = Maximum flexural Service II stress at the midthickness of the flange under consideration

The factored Service II design stresses and forces are shown in Table E24-2.3-15 below.

		Service II Limit State		
Load Case	Location	F_s (ksi)	A_{gross} (in^2)	P_s (kips)
Dead + Pos. LL	Bot. Flange	13.18	12.25	161.49
	Top Flange	0.71	10.50	7.46
Dead + Neg. LL	Bot. Flange	-12.33	12.25	-151.06
	Top Flange	1.94	10.50	20.41

Table E24-2.3-15
Service II Flange Forces

It is important to note here that the flange slip resistance must exceed the larger of: (1) the Service II flange forces or (2) the factored flange forces from the moments at the splice due to constructibility (erection and/or deck pouring sequence) **LRFD [6.13.6.1.4a]**. However, in this design example, no special erection procedure is prescribed, therefore, the deck is assumed to be placed in a single pour. The constructibility moment is then equal to the noncomposite dead load moment shown at the beginning of this design step. By inspection, the Service II Limit State will control for checking of slip-critical connections for the flanges and the web in this example.

Fatigue Limit State stresses:

The final portion of this design step is to determine the range of the stresses at the midthickness of both flanges, and at the top and bottom of the web for the Fatigue Limit State. The ranges are calculated below and presented in Table E24-2.3-16.

A typical calculation of the stress range for the bottom flange is shown below.

From Tables E24-2.3-7 and E24-2.3-8, the factored stresses at the midthickness of the bottom flange are:

Case 1 - Positive Live Load:

$f_{spos} := 5.01$

Case 2 - Negative Live Load:

$f_{sneg} := -3.8$

The stress range is determined by:

$\Delta f := |f_{spos}| + |f_{sneg}|$ $\boxed{\Delta f = 8.81}$ ksi

Location	Fatigue Limit State Stress Range (ksi)
Bottom Flange	8.81
Top Flange	0.42
Bottom of Web	8.73
Top of Web	0.35

Table E24-2.3-16
Fatigue Stress Ranges

E24-2.4 - Design Bottom Flange Splice

Splice plate dimensions:

The width of the outside plate should be at least as wide as the width of the narrowest flange at the splice. Therefore, try a 7/16" x 14" outside splice plate with two 1/2" x 6" inside splice plates. Include a 1/2" x 14" fill plate on the outside. Figure E24-2.4-1 illustrates the initial bottom flange splice configuration.

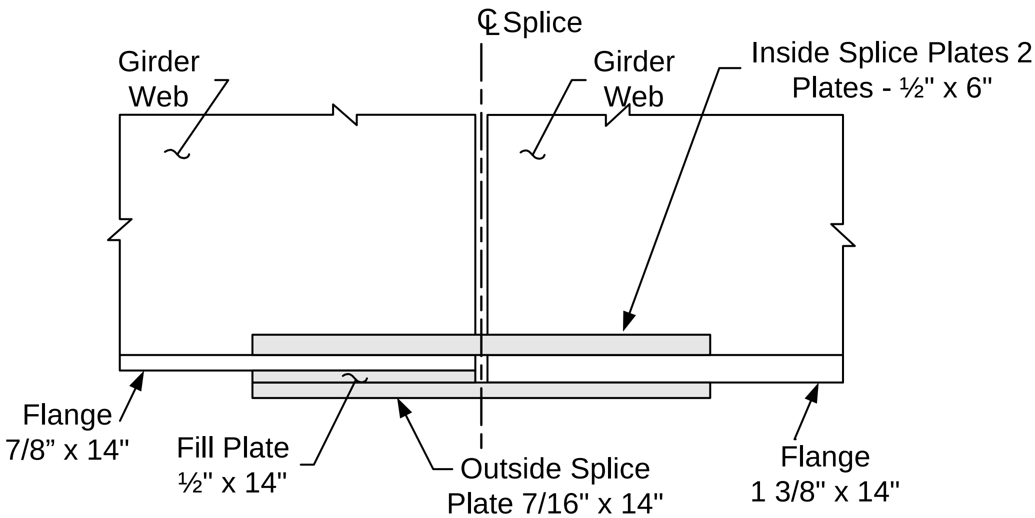

Figure E24-2.4-1
Bottom Flange Splice

The dimensions of the elements involved in the bottom flange splice from Figure E24-2.4-1 are:

Thickness of the inside splice plate:

$t_{in} := 0.50$ in

Width of the inside splice plate:

$b_{in} := 6$ in

Thickness of the outside splice plate:

$t_{out} := 0.4375$ in

Width of the outside splice plate:

$b_{out} := 14$ in

Thickness of the fill plate:

$t_{fill} := 0.50$ in

Width of the fill plate:

$b_{fill} := 14$ in

If the combined area of the inside splice plates is within ten percent of the area of the outside splice plate, then both the inside and outside splice plates may be designed for one-half the flange design force **LRFD [C6.13.6.4.1c]**.

Gross area of the inside and outside splice plates:

Inside:

$A_{gross\_in} := 2 \cdot t_{in} \cdot b_{in}$ $\boxed{A_{gross\_in} = 6.00}$ in^2

Outside:

$A_{gross\_out} := t_{out} \cdot b_{out}$ $\boxed{A_{gross\_out} = 6.13}$ in^2

Check:

$$\left(1 - \frac{A_{gross\_in}}{A_{gross\_out}}\right) \cdot 100\% = 2.04 \cdot \%$$

The combined areas are within ten percent.

If the areas of the inside and outside splice plates had differed by more than ten percent, the flange design force would be proportioned to the inside and outside splice plates. This is calculated by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates.

Yielding and fracture of splice plates:

Case 1 - Tension **LRFD [6.13.5.2]**:

At the Strength Limit State, the design force in the splice plates subjected to tension shall not exceed the factored resistances for yielding, fracture, and block shear.

From Table E24-2.3-13, the Strength I bottom flange tension design force is:

$P_{cu} = 359.21$ kips

The factored tensile resistance for yielding on the gross section, P_r, is taken from **LRFD [6.8.2.1]**:

$P_r := \phi_y \cdot P_{ny}$

Where:

P_{ny} = Nominal tensile resistance for yielding in gross section (kips)

= $F_y A_g$

F_y = Specified minimum yield strength (ksi)

A_g = Gross cross-sectional area of the member (in^2)

$P_r := \phi_y \cdot F_y \cdot A_g$

$F_y = 50.00$ ksi See E24-2.1

$\phi_y = 0.95$ See E24-2.1

For yielding of the outside splice plate:

$A_g := A_{gross\_out}$ $\boxed{A_g = 6.13}$ in^2

$P_r := \phi_y \cdot F_y \cdot A_g$ $\boxed{P_r = 290.94}$ kips

The outside splice plate takes half of the design load:

$P_r = 290.94$ > $\boxed{\frac{P_{cu}}{2} = 179.61}$ OK

For yielding of the inside splice plates:

$A_g := A_{gross\_in}$ $\boxed{A_g = 6.00}$ in^2

$P_r := \phi_y \cdot F_y \cdot A_g$ $\boxed{P_r = 285.00}$ kips

The inside splice plate takes half of the design load:

$P_r = 285.00$ kips > $\boxed{\frac{P_{cu}}{2} = 179.61}$ kips OK

The factored tensile resistance for fracture on the net section, P_r, is calculated by:

$P_r := \phi_u \cdot P_{nu}$

Where:

P_{nu} = Nominal tensile resistance for fracture in net section (kips)

= $F_u A_n R_p U$

F_u = Tensile strength (ksi)

R_p = Reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size.

A_n = Net area of the member (in^2) **LRFD [6.8.3]**

U = reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements, and as specified in **LRFD [6.8.2.2]** for other cases

$P_r := \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U$

$F_u = 65.00$ ksi See E24-2.1

$\phi_u = 0.80$ See E24-2.1

$U := 1.0$

$R_p := 1.0$

To compute the net area of the splice plates, assume four 7/8" bolts across the width of the splice plate.

The net width shall be determined for each chain of holes extending across the member along any transverse, diagonal or zigzag line. This is determined by subtracting from the width of the element the sum of the width of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain. For non-staggered holes, such as in this design example, the minimum net width is the width of the element minus the number of bolt holes in a line straight across the width **LRFD [6.8.3]**.

For fracture of the outside splice plate:

The net width is:

$d_{hole} = 1.00$ in (see E24-2.1)

$b_{n\_out} := b_{out} - 4 \cdot d_{hole}$ $\boxed{b_{n\_out} = 10.00}$ in

The nominal area is determined to be:

$A_{n\_out} := b_{n\_out} \cdot t_{out}$ $\boxed{A_{n\_out} = 4.38}$ in^2

The net area of the connecting element is limited to $0.85A_g$ **LRFD [6.13.5.2]**:

$A_n \leq 0.85 \cdot A_g$

$A_{gross\_out} = 6.13$ in^2

$A_{n\_out} = 4.38$ in^2 < $\boxed{0.85 \cdot A_{gross\_out} = 5.21}$ in^2 OK

$P_r := \phi_u \cdot F_u \cdot A_{n\_out} \cdot U$ $\boxed{P_r = 227.50}$ kips

The outside splice plate takes half of the design flange force:

$P_r = 227.50$ kips > $\boxed{\frac{P_{cu}}{2} = 179.61}$ kips OK

For fracture of the inside splice plates:

The net width is:

$b_{n\_in} := b_{in} - 2 \cdot d_{hole}$ $\boxed{b_{n\_in} = 4.00}$ in

The nominal area is determined to be:

$A_{n\_in} := 2(b_{n\_in} \cdot t_{in})$ $\boxed{A_{n\_in} = 4.00}$ in^2

The net area of the connecting element is limited to $0.85A_g$:

$A_n \le 0.85 \cdot A_g$

$A_{gross\_in} = 6.00$ in^2

$A_{n\_in} = 4.00$ in^2 < $\boxed{0.85 \cdot A_{gross\_in} = 5.10}$ in^2 OK

$P_r := \phi_u \cdot F_u \cdot A_{n\_in} \cdot U$ $\boxed{P_r = 208.00}$ kips

The inside splice plates take half of the design flange force:

$P_r = 208.00$ kips > $\boxed{\frac{P_{cu}}{2} = 179.61}$ kips OK

Case 2 - Compression **LRFD [6.13.6.1.4c]**:

From Table E24-2.3-13, the Strength I bottom flange compression design force is:

$P_{cu} := 459.38$ kips

This force is distributed equally to the inside and outside splice plates.

The factored resistance of the splice plate, R_r, is calculated from:

$R_r := \phi_c \cdot F_y \cdot A_s$

Where:

ϕ_c = resistance factor for compression **LRFD [6.5.4.2]**

F_y = Specified minimum yield strength of the splice plate (ksi)

A_s = Gross area of the splice plate (in^2)

$\phi_c = 0.90$ (see E24-2.1)

For yielding of the outside splice plate:

$A_s := A_{gross\_out}$ $\boxed{A_s = 6.13}$ in^2

$R_{r\_out} := \phi_c \cdot F_y \cdot A_s$ $\boxed{R_{r\_out} = 275.63}$ kips

$R_{r\_out} = 275.63$ kips > $\boxed{\frac{P_{cu}}{2} = 229.69}$ kips OK

For yielding of the inside splice plates:

$A_s := A_{gross\_in}$ $\boxed{A_s = 6.00}$ in^2

$R_{r\_in} := \phi_c \cdot F_y \cdot A_s$ $\boxed{R_{r\_in} = 270.00}$ kips

$R_{r\_in} = 270.00$ kips > $\boxed{\frac{P_{cu}}{2} = 229.69}$ kips OK

Block shear **LRFD [6.13.6.1.4c, 6.13.5.2 & 6.13.4]**:

All tension connections, including connection plates, splice plates and gusset plates, shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection. Block shear rupture will usually not govern the design of splice plates of typical proportion. However, the block shear checks are carried out here for completeness.

From Table E24-2.3-13, the Strength I bottom flange tension design force is:

$P_{cu} := 359.25$ kips

$$R_r := \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_u \cdot A_{tn}) \le \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_y \cdot A_{vg} + U_{bs} \cdot F_u \cdot A_{tn})$$

Where:

R_p = Reduction factor (1.0)

ϕ_{bs} = Resistance factor for block shear **LRFD [6.5.4.2]**

F_y = Specified minimum yield strength of the connected material (ksi)

A_{vg} = Gross area along the plane resisting shear stress (in^2)

F_u = Specified minimum tensile strength of the connected material (ksi) **LRFD [Table 6.4.1-1]**

A_{tn} = Net area along the plane resising tension stress (in^2)

A_{vn} = Net area along the plane resisting shear stress (in^2)

U_{bs} = Reduction factor for block shear rupture resistance taken equal to 0.50 when the tension stress is non-uniform and 1.0 when the tension stress is uniform

From E24-2.1:

$F_y = 50.00$ ksi

$F_u = 65.00$ ksi

$\phi_{bs} = 0.80$

Outside splice plate:

Failure mode 1:

A bolt pattern must be assumed prior to checking an assumed block shear failure mode. An initial bolt pattern for the bottom flange splice, along with the first assumed failure mode, is shown in Figure E24-2.4-2. The outside splice plate will now be checked for block shear.

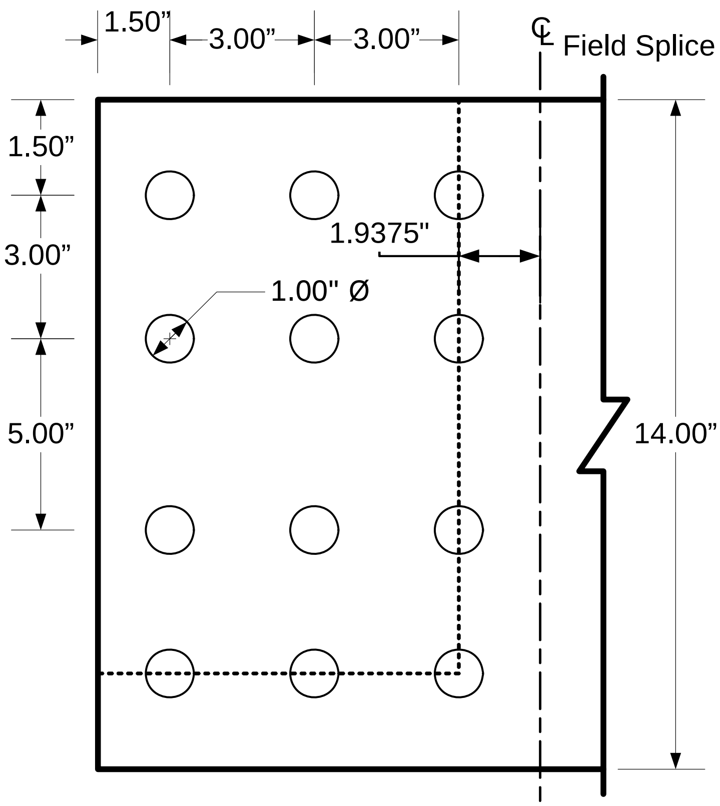

Figure E24-2.4-2
Outside Splice Plate - Failure Mode 1

Applying the factored resistance equations presented previously to the outside splice plate for failure mode 1:

Gross area along the plane resisting shear stress:

$A_{vg} := [2\cdot(3.00) + 1.50]\cdot t_{out}$ $\boxed{A_{vg} = 3.28}$ in^2

Net area along the plane resisting shear stress:

$A_{vn} := \left[2\cdot(3.00) + 1.50 - 2.5\cdot d_{hole}\right]\cdot t_{out}$ $\boxed{A_{vn} = 2.19}$ in^2

Net area along the plane resisting tension stress:

$A_{tn} := \left[[2\cdot(3.00) + 5.00 + 1.50] - 3.5\cdot d_{hole}\right]\cdot t_{out}$ $\boxed{A_{tn} = 3.94}$ in^2

$U_{bs} := 1.0$

$R_{r1} := \phi_{bs}\cdot R_p\cdot\left(0.58\cdot F_u\cdot A_{vn} + U_{bs}\cdot F_u\cdot A_{tn}\right) = 270.73$

$R_{r2} := \phi_{bs}\cdot R_p\cdot\left(0.58\cdot F_y\cdot A_{vg} + U_{bs}\cdot F_u\cdot A_{tn}\right) = 280.88$

$R_r := \min\left(R_{r1}, R_{r2}\right)$

$\boxed{R_r = 270.73}$ kips

Check:

$R_r = 270.73$ kips $>$ $\boxed{\frac{P_{cu}}{2} = 179.63}$ kips OK

Failure mode 2:

See Figure E24-2.4-3 for failure mode 2:

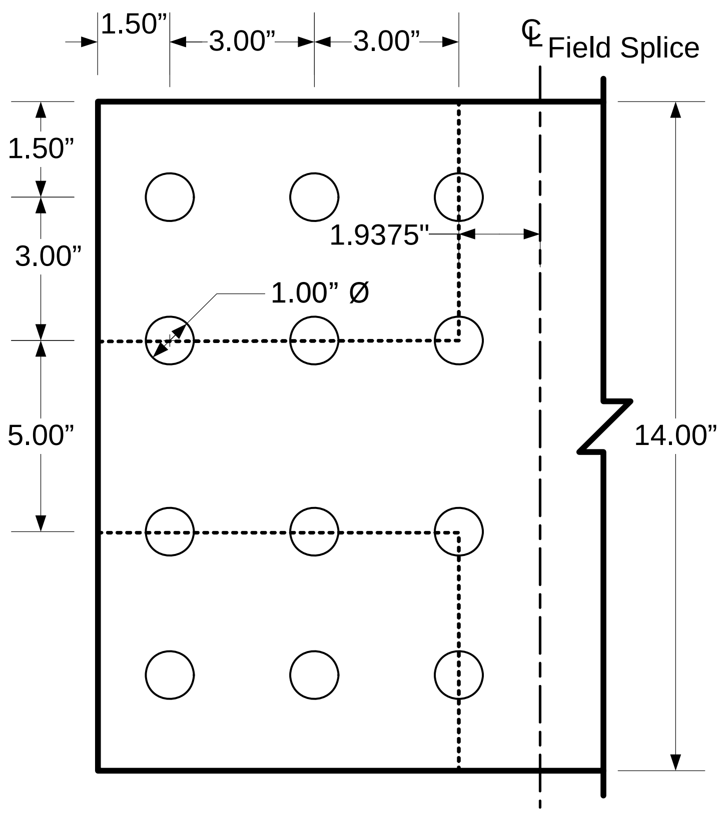

Figure E24-2.4-3
Outside Splice Plate - Failure Mode 2

The failure mode 2 calculations for the outside splice plates are are not shown since they are similar to those shown previously for failure mode 1. The final check for failure mode 2 is shown below.

Check:

$$R_r = 268.45 \text{ kips} > \boxed{\frac{P_{cu}}{2} = 179.63} \text{ kips} \quad \text{OK}$$

Inside splice plates:

The inside splice plates will now be checked for block shear. See Figure E24-2.4-4 for the assumed failure mode:

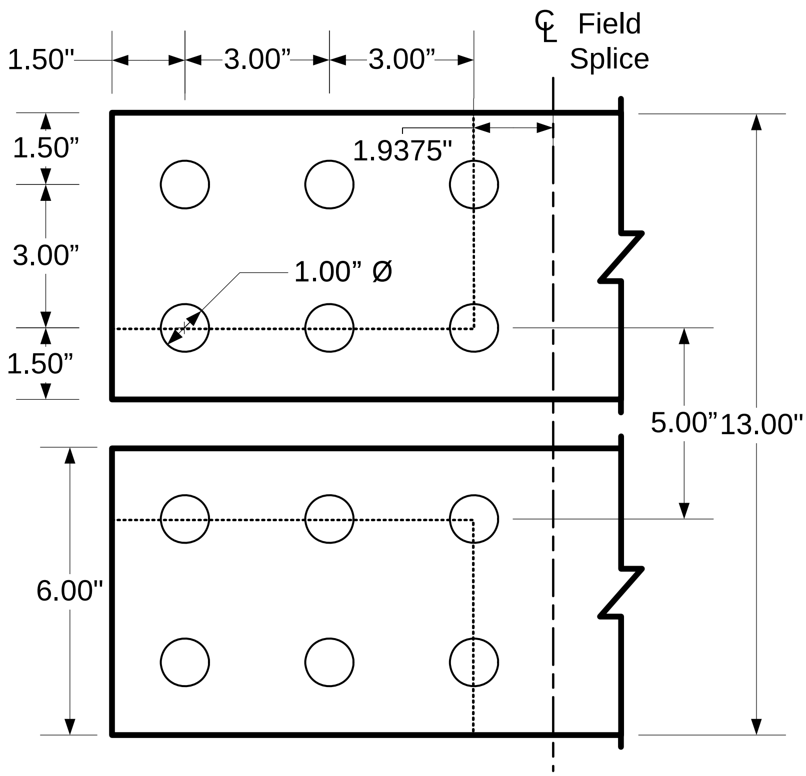

Figure E24-2.4-4
Inside Splice Plates - Block Shear Check

The calculations for the inside splice plates are not shown since they are similar to those shown previously for failure mode 1 and 2. The final check for the inside splice plates is shown below.

Check:

$$R_r = 306.80 \text{ kips} > \boxed{\frac{P_{cu}}{2} = 179.63} \text{ kips} \quad \text{OK}$$

Girder bottom flange:

The girder bottom flange will now be checked for block shear. See Figure E24-2.4-5 for the assumed failure mode:

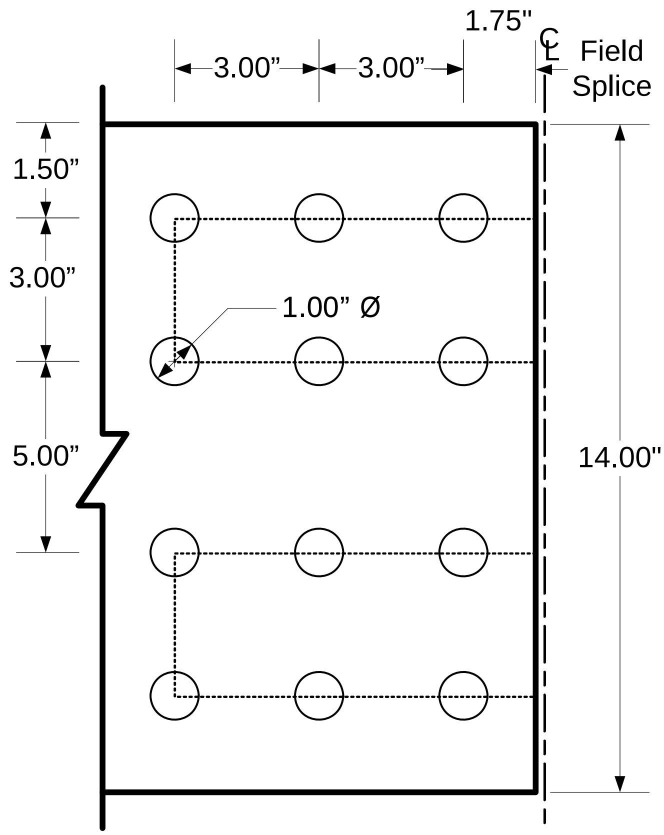

Figure E24-2.4-5
Bottom Flange - Block Shear Check

The calculations for the girder bottom flange are not shown since they are similar to those shown previously for the inside splice plates. The final check for the girder bottom flange is shown below.

Check:

$R_r = 736.19$ kips $>$ $P_{cu} = 359.25$ kips OK

It should be noted that although the block shear checks performed in this design example indicate an overdesign, the number of bolts cannot be reduced prior to checking shear on the bolts and bearing at the bolt holes. These checks are performed in what follows.

Net Section Fracture **LRFD [6.10.1.8]**

When checking flexural members at the strength limit state or for constructibility, the stress on the gross area of the tension flange due to the factored loads calculated without consideration of flange lateral bending, f_t, shall be satisfied at all cross-sections containing holes in the tension flange:

$$f_t \le 0.84 \cdot \left(\frac{A_n}{A_g}\right) \cdot F_u \le F_{yt}$$

Where:

A_n = Net area of the tension flange (in^2) **LRFD[6.8.3]**

A_g = Gross area of the tension flange (in^2)

F_{yt} = Specified minimum yield strength of a tension flange (ksi) **LRFD [C6.8.2.3]**

$A_n := (b_{flbL} - 4 \cdot d_{hole}) \cdot t_{flbL}$ $\boxed{A_n = 8.75}$ in^2

$A_g := t_{flbL} \cdot b_{flbL}$ $\boxed{A_g = 12.25}$ in^2

$F_u = 65.00$ ksi

$\boxed{0.84 \cdot \left(\frac{A_n}{A_g}\right) \cdot F_u = 39.00}$ ksi

$F_{yt} := 50$ ksi

$f_t := 37.5$ ksi (from Table E24-2.3-13)

$f_t \le 0.84 \cdot \left(\frac{A_n}{A_g}\right) \cdot F_u \le f_{yt}$ OK

Flange bolts - shear:

Determine the number of bolts for the bottom flange splice plates that are required to develop the Strength I design force in the flange in shear assuming the bolts in the connection have slipped and gone into bearing. A minimum of two rows of bolts should be provided to ensure proper alignment and stability of the girder during construction.

The Strength I flange design force used in this check was previously computed (reference Table E24-2.3-13):

$P_{cu} := 459.38$ kips

The factored resistance of an ASTM A325 7/8" diameter high-strength bolt in shear must be determined, assuming the threads are excluded from the shear planes. For this case, the number of bolts required to provide adequate shear strength is determined by assuming the design force acts on two shear planes, known as double shear.

The nominal shear resistance, R_n, is computed first as follows **LRFD [6.13.2.7]**:

$$R_n := (0.48 \cdot A_b \cdot F_{ub} \cdot N_s)$$

Where:

A_b = Area of the bolt corresponding to the nominal diameter (in^2)

F_{ub} = Specified minimum tensile strength of the bolt (ksi) **LRFD [6.4.3]**

N_s = Number of shear planes per bolt

$A_b := \frac{\pi}{4} \cdot d_{bolt}^{\,2}$ $\boxed{A_b = 0.60}$ in^2

$F_{ub} := Fu_{bolt}$ $\boxed{F_{ub} = 120.00}$ ksi

$N_s := 2$

$R_n := 2 \cdot (0.48 \cdot A_b \cdot F_{ub})$ $\boxed{R_n = 69.27}$ kips

The factored shear resistance now follows:

$\phi_s = 0.80$ (see E24-2.1)

$R_u := \phi_s \cdot R_n$ $\boxed{R_u = 55.42}$ kips

When bolts carrying loads pass through fillers 0.25 inches or more in thickness in axially loaded connections, including girder flange splices, either **LRFD [6.13.6.1.5]**:

The fillers shall be extended beyond the gusset or splice material and shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler.

or

The fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the Strength Limit State, specified in **LRFD [6.13.2.2]**, is reduced by an appropriate factor:

In this design example, the reduction factor approach will be used. The reduction factor, R, per the specifications is:

$$R := \left(\frac{1+\gamma}{1+2\gamma}\right)$$

Where:

γ = A_f / A_p

A_f = Sum of the area of the fillers on the top and bottom of the connected plate (in^2)

A_p = Smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in^2)

Sum of the area of the fillers on the top and bottom of the connected plate:

$A_f := b_{fill} t_{fill}$ $\boxed{A_f = 7.00}$ in^2

The smaller of either the connected plate area (i.e., girder flange) or the sum of the splice plate areas on the top and bottom of the connected plate determines A_p.

Bottom flange area:

$b_{flbL} = 14.00$ in

$t_{flbL} = 0.88$ in

$A_{p1} := (b_{flbL}) \cdot (t_{flbL})$ $\boxed{A_{p1} = 12.25}$ in^2

Sum of splice plate areas is equal to the gross areas of the inside and outside splice plates:

$A_{gross\_in} = 6.00$ in

$A_{gross\_out} = 6.13$ in

$A_{p2} := A_{gross\_in} + A_{gross\_out}$ $\boxed{A_{p2} = 12.13}$ in^2

The minimum of the areas is:

$A_p := \min(A_{p1}, A_{p2})$ $\boxed{A_p = 12.13}$ in^2

Therefore:

$\gamma := \frac{A_f}{A_p}$ $\boxed{\gamma = 0.58}$

The reduction factor is determined to be:

$R_{fill} := \left(\frac{1 + \gamma}{1 + 2\gamma}\right)$ $\boxed{R_{fill} = 0.73}$

To determine the total number of bolts required for the bottom flange splice, divide the applied Strength I flange design force by the reduced allowable bolt shear strength:

$R := R_u \cdot R_{fill}$ $\boxed{R = 40.57}$ kips

The number of bolts required per side is:

$N := \frac{P_{cu}}{R}$ $\boxed{N = 11.32}$

The minimum number of bolts required on each side of the splice to resist the maximum Strength I flange design force in shear is twelve.

Flange bolts - slip resistance:

Bolted connections for flange splices shall be designed as slip-critical connections for the Service II flange design force, or the flange design force from constructibility, whichever governs **LRFD [6.13.6.1.4a]**. In this design example, the Service II flange force controls (see E24-2.3).

When checking for slip of the bolted connection for a flange splice with inner and outer splice plates, the slip resistance should always be determined by dividing the flange design force equally to the two slip planes regardless of the ratio of the splice plate areas **LRFD [C6.13.6.1.4c]**. Slip of the connection cannot occur unless slip occurs on both planes.

From Table E24-2.3-15, the Service II bottom flange design force is:

$P_s := 161.49$ kips

The factored resistance for slip-critical connections, R_n, is calculated from **LRFD [6.13.2.2 & 6.13.2.8]**:

$R_r := R_n$

$R_n := K_h \cdot K_s \cdot N_s \cdot P_t$

Where:

K_h = Hole size factor **LRFD [Table 6.13.2.8-2]**

K_s = Surface condition factor **LRFD [Table 6.13.2.8-3]**

N_s = Number of slip planes per bolt

P_t = Minimum required bolt tension (kips) **LRFD [Table 6.13.2.8-1]**

Determine the factored resistance per bolt assuming a Class B surface condition for the faying surface, standard holes (which are required per **LRFD [6.13.6.1.4a]**) and two slip planes per bolt:

Class B surfaces are unpainted blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings **LRFD [6.13.2.8]**.

$K_h := 1.0$

$K_s := 0.50$

$N_s := 2$

$P_t := 39.0$ kips

$R_r := K_h \cdot K_s \cdot N_s \cdot P_t$ $\boxed{R_r = 39.00}$ kips

The minimum number of bolts required to prevent slip is:

$$N := \frac{P_s}{R_r} \qquad \boxed{N = 4.14}$$

Use:

$N := 5$ bolts < N = 12 bolts determined previously to satisfy the bolt shear requirements.

Therefore, the number of bolts required for the bottom-flange splice is controlled by the bolt shear requirements. Arrange the bolts in three rows of four bolts per line with no stagger.

Flange bolts - minimum spacing **LRFD [6.13.2.6.1]**:

The minimum spacing between centers of bolts in standard holes shall be no less than three times the diameter of the bolt.

$d_{bolt} = 0.875$ in

$s_{min} := 3 \cdot d_{bolt}$ $\boxed{s_{min} = 2.63}$ in

$s := 3.00$ in (see Figures E24-2.4-2 thru E24-2.4-5)

The minimum spacing requirement is satisfied.

Flange bolts - maximum spacing for sealing **LRFD [6.13.2.6.2]**:

For a single line adjacent to a free edge of an outside plate or shape (for example, the bolts along the edges of the plate parallel to the direction of the applied force):

$$s \le (4.0 + 4.0 \cdot t) \le 7.0$$

Where:

t = Thickness of the thinner outside plate or shape (in)

$t_{out} = 0.4375$ in

Maximum spacing for sealing:

$\boxed{4.0 + 4.0 \cdot t_{out} = 5.75}$ in

$s \le 5.75 \le 7.00$ OK

Next, check for sealing along the free edge at the end of the splice plate. The bolts are not staggered, therefore the applicable equation is:

$$s \le (4.00 + 4.00 \cdot t) \le 7.00$$

Maximum spacing along the free edge at the end of the splice plate (see Figures E24-2.4-2 thru E24-2.4-5):

$s_{end} := 5.00$ in

Maximum spacing for sealing:

$\boxed{4.0 + 4.0 \cdot t_{out} = 5.75}$ in

$s_{end} \leq 5.75 \leq 7.00$ OK

Flange bolts - maximum pitch for stitch bolts **LRFD [6.13.2.6.3]**:

The maximum pitch requirements are applicable only for mechanically fastened built-up members and will not be applied in this example.

Flange bolts - edge distance **LRFD [6.13.2.6.6]**:

The minimum required edge distance is measured as the distance from the center of any bolt in a standard hole to an edge of the plate. For a 7/8" diameter bolt measured to a sheared edge, the minimum edge distance is 1 1/2" **LRFD [Table 6.13.2.6.6-1]**. Referring to Figures E24-2.4-2 thru E24-2.4-5, it is clear that the minimum edge distance specified for this example is 1 1/2" and thus satisfies the minimum requirement.

The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or five inches.

$8 \cdot t \leq 5.00$ in

$t := t_{out}$ $\boxed{t_{out} = 0.4375}$ in

$\boxed{8 \cdot t_{out} = 3.50}$ in

The maximum distance from the corner bolts to the corner of the splice plate or girder flange is equal to (reference Figure E24-2.4-5):

$\boxed{\sqrt{1.50^2 + 1.75^2} = 2.30}$ in

$2.30 \cdot in \leq 3.50 \cdot in$ OK

Flange bolts - bearing at bolt holes **LRFD [6.13.2.9]**:

Check bearing of the bolts on the connected material under the maximum Strength I Limit State design force. The maximum Strength I bottom flange design force from Table E24-2.3-13 is:

$P_{cu} := 459.38$

The design bearing strength of the connected material is calculated as the sum of the bearing strengths of the individual bolt holes parallel to the line of the applied force.

The element of the bottom flange splice that controls the bearing check in this design example is the outer splice plate.

To determine the applicable equation for the calculation of the nominal resistance, the clear distance between holes and the clear end distance must be calculated and compared to the value of two times the nominal diameter of the bolt. This check yields:

$d_{bolt} = 0.875$ in $\boxed{2 \cdot d_{bolt} = 1.75}$ in

For the bolts adjacent to the end of the splice plate, the edge distance is 1 1/2". Therefore, the clear end distance between the edge of the hole and the end of the splice plate:

$d_{hole} = 1.00$

$L_{c_1} := 1.50 - \frac{d_{hole}}{2}$ $\boxed{L_{c_1} = 1.00}$ in

The center-to-center distance between bolts in the direction of the force is three inches. Therefore, the clear distance between edges of adjacent holes is computed as:

$L_{c_2} := 3.00 - d_{hole}$ $\boxed{L_{c_2} = 2.00}$ in

For standard holes, where either the clear distance between holes or the clear end distance is less than twice the bolt diameter, the nominal resistance, R_n, is taken as:

$R_n := 1.2 \cdot L_c \cdot t \cdot F_u$

Where:

L_c = Clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in)

For the outside splice plate:

$t_{out} = 0.4375$ in

$F_u = 65.00$ in

The nominal resistance for the end row of bolt holes is computed as follows:

$R_{n_1} := 4 \cdot \left(1.2 \cdot L_{c_1} \cdot t_{out} \cdot F_u\right)$ $\boxed{R_{n_1} = 136.50}$ kips

The nominal resistance for the remaining bolt holes is computed as follows:

$R_{n_2} := 8 \cdot \left(1.2 \cdot L_{c_2} \cdot t_{out} \cdot F_u\right)$ $\boxed{R_{n_2} = 546.00}$ kips

The total nominal resistance of the bolt holes is:

$R_n := R_{n_1} + R_{n_2}$ $\boxed{R_n = 682.50}$ kips

$\phi_{bb} = 0.80$

$R_r := \phi_{bb} \cdot R_n$ $\boxed{R_r = 546.00}$ kips

Check:

$\boxed{\frac{P_{cu}}{2} = 229.69}$ kips < $R_r = 546.00$ kips OK

Fatigue of splice plates **LRFD [6.6.1]**:

Check the fatigue stresses in the base metal of the bottom flange splice plates adjacent to the slip-critical connections. Fatigue normally does not govern the design of the splice plates, and therefore, an explicit check is not specified. However, a fatigue check of the splice plates is recommended whenever the combined area of the inside and outside flange splice plates is less than the area of the smaller flange at the splice.

From Table E24-2.3-16, the factored fatigue stress range at the midthickness of the bottom flange is:

$\Delta f_{fact} := 8.81$ ksi

For load-induced fatigue considerations, each detail shall satisfy:

$\gamma \cdot (\Delta f) \leq \Delta F_n$

Where:

γ = Load factor for fatigue I load combination **LRFD [Table 3.4.1-1]**

(Δf) = Force effect, live load stress range due to the passage of the fatigue load (ksi) **LRFD [3.6.1.4]**

ΔF_n = Nominal fatigue resistance (ksi) **LRFD [6.6.1.2.5]**

$\gamma := 1.50$

$\gamma(\Delta f) := \Delta f_{fact}$ $\boxed{\gamma(\Delta f) = 8.81}$ ksi

For the fatigue I load combination:

$\Delta F_n = \blacksquare \cdot \Delta F_{TH}$

Where:

ΔF_{TH} = Constant-amplitude fatigue threshold (ksi) **LRFD [Table 6.6.1.2.5-3]**

$\Delta F_{TH} := 16$ ksi

$\Delta F_n := \Delta F_{TH}$ $\boxed{\Delta F_n = 16.00}$ ksi (governs)

Check that the following is satisfied:

$\Delta f_{fact} \leq \Delta F_n$ $\Delta f_{fact} = 8.81$ ksi < $\Delta F_n = 16.00$ ksi OK

Control of permanent deflection - splice plates **LRFD [6.10.4.2]**:

A check of the flexural stresses in the splice plates at the Service II Limit State is not explicitly specified in the specifications. However, whenever the combined area of the inside and outside flange splice plates is less than the area of the smaller flange at the splice (which is the case for the bottom flange splice in this example), such a check is recommended.

The maximum Service II flange force in the bottom flange is taken from Table E24-2.3-15:

$P_S = 161.49$ kips

The following criteria will be used to make this check **LRFD [6.10.4.2]**. The equation presented is for both steel flanges of a composite section assuming no lateral flange bending:

$$f_f \leq 0.95 \cdot R_h \cdot F_{yf}$$

Where:

f_f = Flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending (ksi)

R_h = Hybrid factor **LRFD [6.10.1.10.1]**

F_{yf} = Specified minimum yield strength of a flange (ksi) **LRFD [6.7.7.3]**

$F_{yf} = 50.00$ ksi

The flange force is equally distributed to the inner and outer splice plates due to the areas of the flanges being within 10 percent of each other:

$$P := \frac{P_S}{2} \qquad \boxed{P = 80.75} \text{ kips}$$

The resulting stress in the outside splice plate is:

$$f\_out := \frac{P}{A_{gross\_out}} \qquad \boxed{f\_out = 13.18} \text{ ksi}$$

$$f\_out = 13.18 \text{ ksi} < \boxed{0.95 \cdot F_{yf} = 47.50} \text{ ksi OK}$$

The resulting stress in the inside splice plates is:

$$f\_in := 13.46 \text{ ksi} < \boxed{0.95 \cdot R_h \cdot F_{yf} = 47.50} \text{ ksi OK}$$

E24-2.5 Design Top Flange Splice

The top flange splice is designed using the same procedures and methods presented in this design example for the bottom flange splice.

E24-2.6 Compute Web Splice Design Loads

Web splice plates and their connections shall be designed for shear, the moment due to the eccentricity of the shear at the point of splice, and the portion of the flexural moment assumed to be resisted by the web at the point of the splice **LRFD [6.13.6.1.4b]**.

Girder shear forces at the splice location:

A summary of the unfactored shears at the splice location from the initial trial of the girder design are listed below. The live loads include impact and distribution factors.

Dead load shears:

Noncomposite:

$V_{NDL} := -58.3$ kips

Composite:

$V_{CDL} := -7.7$ kips

Future wearing surface:

$V_{FWS} := -7.3$ kips

Live Load shears:

HL-93 positive:

$V_{PLL} := 14.7$ kips

HL-93 negative:

$V_{NLL} := -94.0$ kips

Fatigue positive:

$V_{PFLL} := 5.2$ kips

Fatigue negative:

$V_{NFLL} := -34.2$ kips

Web moments and horizontal force resultant **LRFD [C6.13.6.1.4b]**:

Because the portion of the flexural moment assumed to be resisted by the web is to be applied at the mid-depth of the web, a horizontal design force resultant, H_{uw}, must also be applied at the mid-depth of the web to maintain equilibrium. The web moment and horizontal force resultant are applied together to yield a combined stress distribution equivalent to the unsymmetrical stress distribution in the web. For sections with equal compressive and tensile stresses at the top and bottom of the web (i.e., with the neutral axis located at the mid-depth of the web), H_{uw} will equal zero.

In the computation of the portion of the flexural moment assumed to be resisted by the web, M_{uw}, and the horizontal design force resultant, H_{uw}, in the web, the flange stresses at the midthickness of the flanges are conservatively used. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations. It is important to note that the flange stresses are taken as signed quantities in determining M_{uw} and H_{uw} (positive for tension; negative for compression).

The moment, M_{uv}, due to the eccentricity of the design shear, V_{uw}, is resisted solely by the web and always acts about the mid-depth of the web (i.e., horizontal force resultant is zero). This moment is computed as:

$$M_{uv} := V_{uw} \cdot e$$

Where:

e = The distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration (in)

$$e := 1.9375 + \frac{3.00}{2}$$ (see Figure E24-2.7-1) $\boxed{e = 3.4375}$ in

The total web moment for each load case is computed as follows:

$$M_{total} := M_{uw} + M_{uv}$$

In general, and in this example, the web splice is designed under the conservative assumption that the maximum moment and shear at the splice will occur under the same loading condition.

Strength I Limit State:

Design shear **LRFD [6.13.6.1.4b]**:

For the Strength I Limit State, the girder web factored shear resistance is required when determining the design shear **LRFD [6.10.9.2]**. Assume an unstiffened web at the splice location. The nominal shear resistance, V_n, is as follows:

$$V_n := C \cdot V_p$$

Where:

C = Ratio of the shear-buckling resistance to the shear yield strength in accordance with **LRFD [6.10.9.3.2]**, with the shear-buckling coefficient, k, taken equal to 5.0

V_p = Plastic shear force (kips)
= $0.58F_{yw}Dt_w$

F_{yw} = Specified minimum yield strength of a web (ksi)

D = Clear distance between flanges (in)

t_w = Web thickness (in)

$k := 5.0$

$E := 29000$ ksi

$F_{yw} := F_y$ $\boxed{F_{yw} = 50.00}$ ksi

$D = 54.00$ in (see Figure E24-2.1-1)

$t_w = 0.50$ in (see Figure E24-2.1-1)

Compare:

$$\boxed{\frac{D}{t_w} = 108.00}$$

to the values for:

$$\boxed{1.12 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 60.31}$$

and

$$\boxed{1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 75.39}$$

Based on the computed value of D/t_w, use the following equation to determine C **LRFD [6.10.9.3.2]**:

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) \qquad \boxed{C = 0.39}$$

The nominal shear resistance, V_n, is computed as follows:

$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w$ $\boxed{V_p = 783.00}$ kips

$V_n := C \cdot V_p$ $\boxed{V_n = 305.64}$ kips

The factored shear resistance, V_r, now follows:

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 305.64}$ kips

At the strength limit state, the design shear, V_{uw}, shall be taken as **LRFD [6.13.6.1.4b]**:

If:

$V_u < 0.5\ \phi_v V_n$, then:

then:

$V_{uw} := 1.5 \cdot V_u$

Otherwise:

$$V_{uw} := \frac{V_u + (\phi_v \cdot V_n)}{2}$$

The shear due to the Strength I loading at the point of splice, V_u, is computed from the girder shear forces at the splice location listed at the beginning of this section.

For the Strength I Limit State, the factored shear for the positive live load is:

$V_{upos} := 0.90 \cdot (V_{NDL} + V_{CDL}) + 1.75 \cdot V_{PLL}$ $\boxed{V_{upos} = -33.67}$ kips

For the Strength I Limit State, the factored shear for the negative live load is:

$V_{uneg} := 1.25 \cdot (V_{NDL} + V_{CDL}) + 1.50 \cdot V_{FWS} + 1.75 \cdot V_{NLL}$

$\boxed{V_{uneg} = -257.95}$ kips (controls)

Therefore:

$V_u := |V_{uneg}|$ $\boxed{V_u = 257.95}$ kips

Since V_u exceeds one-half of $\phi_v V_n$:

$$V_{uw} := \frac{V_u + (\phi_v \cdot V_n)}{2}$$ $\boxed{V_{uw} = 281.80}$ kips

Web moments and horizontal force resultants:

Case 1 - Dead load + positive live load:

For the loading condition with positive live load, the controlling flange was previously determined to be the bottom flange. The maximum elastic flexural stress due to the factored loads at the midthickness of the controlling flange, f_{cf}, and the design stress for the controlling flange, F_{cf}, were previously computed for this loading condition. From Table E24-2.3-13:

$f_{cf} := 18.32$ ksi

$F_{cf} := 37.50$ ksi

For the same loading condition, the concurrent flexural stress at the midthickness of the noncontrolling (top) flange, f_{ncu}, was previously computed. From Table E24-2.3-14:

$f_{ncf} := 0.33$ ksi

Therefore, the portion of the flexural moment, M_{uw}, assumed to be resisted by the web is computed as:

$$M_{uw} := \frac{t_w \cdot D^2}{12} \cdot |R_h \cdot F_{cf} - R_{cf} \cdot f_{ncf}|$$

Where:

F_{cf} = Design stress for the controlling flange at the point of splice specified in **LRFD [6.13.6.1.4c]**; positive for tension, negative for compression (ksi)

R_{cf} = The absolute value of the ratio of Fcf to the maximum flexural stress, fcf, due to the factored loads at the midthickness of the controlling flange at the point of splice, as defined in **LRFD [6.13.6.1.4c]**

f_{ncf} = Flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with f_{cf}; positive for tension, negative for compression (ksi)

$$R_h = 1.00$$

$$R_{cf} := \left|\frac{F_{cf}}{f_{cf}}\right| \qquad \boxed{R_{cf} = 2.05}$$

Compute the portion of the flexural moment to be resisted by the web:

$$M_{uw\_str\_pos} := \frac{t_w \cdot D^2}{12} \cdot \left|R_h \cdot F_{cf} - R_{cf} \cdot f_{ncf}\right| \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{uw\_str\_pos} = 372.85} \quad \text{kip-ft}$$

The total web moment is:

$$M_{tot\_str\_pos} := M_{uw\_str\_pos} + \left(V_{uw} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_str\_pos} = 453.57} \quad \text{kip-ft}$$

Compute the horizontal force resultant (the variables included in this equation are as defined for $M_{w\_str\_pos}$):

$$H_{uw\_str\_pos} := \frac{t_w \cdot D}{2} \cdot \left(R_h \cdot F_{cf} + R_{cf} \cdot f_{ncf}\right) \qquad \boxed{H_{uw\_str\_pos} = 515.37} \quad \text{kips}$$

The above value is a signed quantity, positive for tension and negative for compression.

Case 2 - Dead load + negative live load:

The calculations at the Strength I Limit State for case 2 are not shown since they are similar to those shown previously for case 1. The final web moment and horizontal force resultant for case 2 are shown below.

The total web moment is:

$$M_{tot\_str\_neg} := M_{uw\_str\_neg} + \left(V_{uw} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_str\_neg} = 759.38} \quad \text{kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_str\_neg} := \frac{t_w \cdot D}{2} \cdot \left(R_h \cdot F_{cf} + R_{cf} \cdot f_{ncf}\right) \qquad \boxed{H_{uw\_str\_neg} = -107.62} \text{ kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

Service II Limit State:

Design shear:

As a minimum, for checking slip of the web splice bolts, the design shear shall be taken as the shear at the point of splice under the Service II Limit State, or the shear from constructibility, whichever governs **LRFD [6.13.6.1.4b]**. In this design example, the Service II shear controls (see previous discussion in section E24-2.3).

For the Service II Limit State, the factored shear for the positive live load is (ignore future wearing surface):

$$V_{ser\_pos} := 1.00 \cdot V_{NDL} + 1.00 \cdot V_{CDL} + 1.30 \cdot V_{PLL} \qquad \boxed{V_{ser\_pos} = -46.89} \text{ kips}$$

For the Service II Limit State, the factored shear for the negative live load is (include future wearing surface):

$$V_{ser\_neg} := 1.00 \cdot V_{NDL} + 1.00 \cdot V_{CDL} + 1.00 \cdot V_{FWS} + 1.30 \cdot V_{NLL}$$

$$\boxed{V_{ser\_neg} = -195.50} \text{ kips (governs)}$$

Therefore:

$$V_{w\_ser} := \left|V_{ser\_neg}\right| \qquad \boxed{V_{w\_ser} = 195.50} \text{ kips}$$

Web moments and horizontal force resultants **LRFD [C6.13.6.1.4b]**:

$$M_{uw\_ser} := \frac{t_w \cdot D^2}{12} \cdot \left|f_s - f_{os}\right|$$

$$H_{uw\_ser} := \frac{t_w \cdot D}{2} \cdot \left(f_s + f_{os}\right)$$

Where:

f_s = Maximum Service II midthickness flange stress for the load case considered (i.e., positive or negative live load) (ksi)

f_{os} = Service II midthickness flange stress in the other flange, concurrent with f_s

Case 1 - Dead load + positive live load:

The maximum midthickness flange flexural stress for the load case with positive live load moment for the Service II Limit State occurs in the bottom flange. From Table 2E2.34-11:

$f_{s\_bot\_pos} := 13.18$ ksi

$f_{os\_top\_pos} := 0.71$ ksi

Therefore, for the load case of positive live load:

$$M_{uw\_ser\_pos} := \frac{t_w \cdot D^2}{12} \cdot \left| f_{s\_bot\_pos} - f_{os\_top\_pos} \right| \cdot \left(\frac{1}{12}\right)$$

$$\boxed{M_{uw\_ser\_pos} = 126.26} \quad \text{kip-ft}$$

$$M_{tot\_ser\_pos} := M_{uw\_ser\_pos} + \left(V_{w\_ser} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_ser\_pos} = 182.26} \quad \text{kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_ser\_pos} := \frac{t_w \cdot D}{2} \cdot \left(f_{s\_bot\_pos} + f_{os\_top\_pos}\right) \qquad \boxed{H_{uw\_ser\_pos} = 187.52} \quad \text{kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

Case 2 - Dead load + negative live load:

The maximum midthickness flange flexural stress for the load case with negative live load moment for the Service II Limit State occurs in the bottom flange. From Table E24-2.3-11:

$f_{s\_bot\_neg} := -12.33$ ksi

$f_{os\_top\_neg} := 1.94$ ksi

Therefore:

$$M_{uw\_ser\_neg} := \frac{t_w \cdot D^2}{12} \cdot \left| f_{s\_bot\_neg} - f_{os\_top\_neg} \right| \cdot \left(\frac{1}{12}\right)$$

$$\boxed{M_{uw\_ser\_neg} = 144.48} \quad \text{kip-ft}$$

The total web moment is:

$$M_{tot\_ser\_neg} := M_{uw\_ser\_neg} + \left(V_{w\_ser} \cdot e\right) \cdot \left(\frac{1}{12}\right)$$

$$\boxed{M_{tot\_ser\_neg} = 200.49} \quad \text{kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_ser\_neg} := \frac{t_w \cdot D}{2} \cdot \left(f_{s\_bot\_neg} + f_{os\_top\_neg}\right) \qquad \boxed{H_{uw\_ser\_neg} = -140.27} \quad \text{kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

Fatigue I Limit State:

Fatigue of the base metal adjacent to the slip-critical connections in the splice plates may be checked as specified in **LRFD [Table 6.6.1.2.3-1]** using the gross section of the splice plates and member **LRFD [C6.13.6.1.4a]**. However, the areas of the web splice plates will often equal or exceed the area of the web to which it is attached (the case in this design example). Therefore, fatigue will generally not govern the design of the splice plates, but is carried out in this example for completeness.

Design shear:

For the Fatigue I Limit State, the factored shear for the positive live load is:

$$V_{fat\_pos} := 1.50 \cdot V_{PFLL} \qquad \boxed{V_{fat\_pos} = 7.80} \quad \text{kips}$$

For the Fatigue I Limit State, the factored shear for the negative live load is:

$$V_{fat\_neg} := 1.50 \cdot V_{NFLL} \qquad \boxed{V_{fat\_neg} = -51.30} \quad \text{kips}$$

Web moments and horizontal force resultants:

The portion of the flexural moment to be resisted by the web and the horizontal force resultant are computed from equations similar to **LRFD [Equations C6.13.6.1.4b-1 & 6.13.6.1.4b-2]**, respectively, with appropriate substitutions of the stresses in the web caused by the fatigue-load moment for the flange stresses in the equations. Also, the absolute value signs are removed to keep track of the signs. This yields the following equations:

$$M_{uw} := \frac{t_w \cdot D^2}{12} \cdot \left(f_{botweb} - f_{topweb}\right)$$

$$H_{uw} := \frac{t_w \cdot D}{2} \cdot \left(f_{botweb} + f_{topweb}\right)$$

Case 1 - Positive live load:

The factored stresses due to the positive live load moment for the Fatigue I Limit State at the top and bottom of the web, from Table E24-2.3-12, are:

$$f_{topweb\_pos} := -0.20 \quad \text{ksi}$$

$$f_{botweb\_pos} := 4.96 \quad \text{ksi}$$

Therefore:

$$M_{uw\_fat\_pos} := \frac{t_w \cdot D^2}{12} \cdot \left(f_{botweb\_pos} - f_{topweb\_pos}\right) \cdot \left(\frac{1}{12}\right)$$

$$\boxed{M_{uw\_fat\_pos} = 52.25} \quad \text{kip-ft}$$

The total web moment is:

$$M_{tot\_fat\_pos} := M_{uw\_fat\_pos} + \left(V_{fat\_pos} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_fat\_pos} = 54.48} \text{ kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_fat\_pos} := \frac{t_w \cdot D}{2} \cdot \left(f_{botweb\_pos} + f_{topweb\_pos}\right) \qquad \boxed{H_{uw\_fat\_pos} = 64.26} \text{ kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

Case 2 - Negative live load:

The factored stresses due to the negative live load moment for the Fatigue I Limit State at the top and bottom of the web, from Table E24-2.3-12, are:

$f_{botweb\_neg} := -3.77$ ksi

$f_{topweb\_neg} := 0.15$ ksi

Therefore:

$$M_{uw\_fat\_neg} := \frac{t_w \cdot D^2}{12} \cdot \left(f_{botweb\_neg} - f_{topweb\_neg}\right) \cdot \left(\frac{1}{12}\right)$$

$$\boxed{M_{uw\_fat\_neg} = -39.69} \text{ kip-ft}$$

The total web moment is:

$$M_{tot\_fat\_neg} := M_{uw\_fat\_neg} + \left(V_{fat\_neg} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_fat\_neg} = -54.39} \text{ kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_fat\_neg} := \frac{t_w \cdot D}{2} \cdot \left(f_{botweb\_neg} + f_{topweb\_neg}\right) \qquad \boxed{H_{uw\_fat\_neg} = -48.87} \text{ kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

E24-2.7 Design Web Splice

Web splice configuration:

Two vertical rows of bolts with sixteen bolts per row will be investigated. The typical bolt spacings, both horizontally and vertically, are as shown in Figure E24-2.7-1. The outermost rows of bolts are located 4 1/2" from the flanges to provide clearance for assembly (see the *AISC Manual of Steel Construction* for required bolt assembly clearances). The web is spliced symmetrically by plates on each side with a thickness not less than one-half the thickness of the web. Assume 3/8" x 48" splice plates on each side of the web. No web fill plate is necessary for this example.

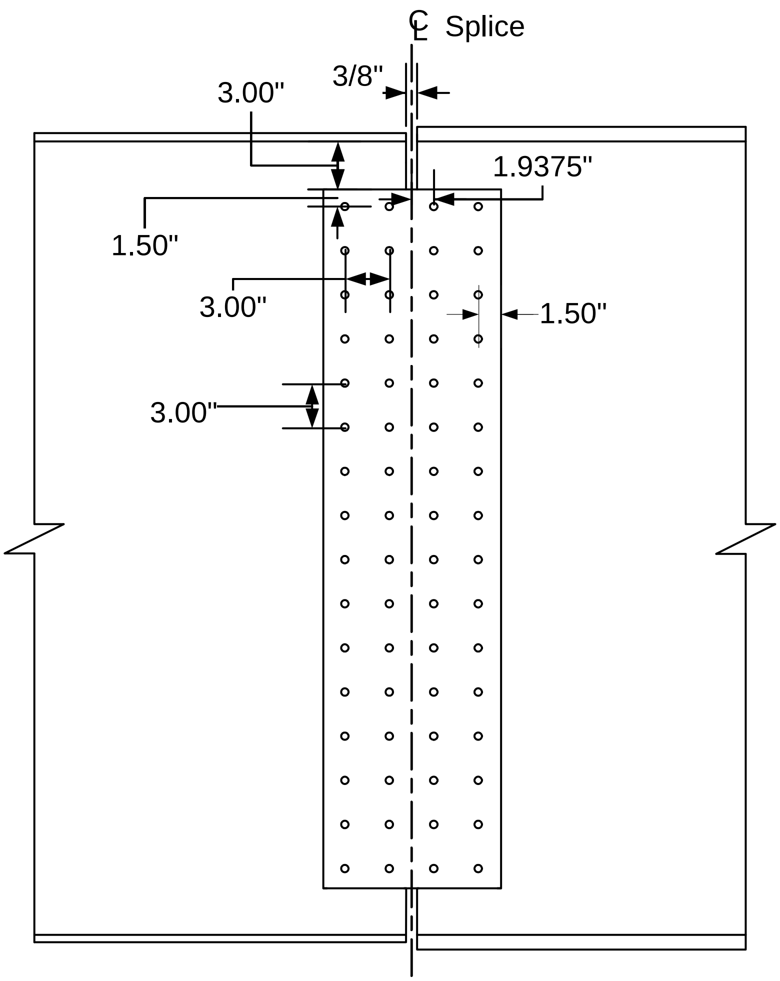

Figure E24-2.7-1
Web Splice

Web bolts - minimum spacing **LRFD [6.13.2.6.1]**:

This check is only dependent upon the bolt diameter, and is therefore satisfied for a three inch spacing per the check for the flange bolts from E24-2.4.

Web Bolts - Maximum Spacing for Sealing **LRFD [6.13.2.6.2]**:

The maximum spacing of the bolts is limited to prevent penetration of moisture in the joints.

For a single line adjacent to a free edge of an outside plate or shape (for example, the bolts along the edges of the plate parallel to the direction of the applied force):

$s \le (4.0 + 4.0 \cdot t) \le 7.0$

$t_{wp} := 0.375$ in

Maximum spacing for sealing:

$\boxed{4.0 + 4.0 \cdot t_{wp} = 5.50}$ in

$3.0 \le 5.5 \le 7.00$ OK

Web bolts - maximum pitch for stitch bolts **LRFD [6.13.2.6.3]**:

The maximum pitch requirements are applicable only for mechanically fastened built-up members and will not be applied in this example.

Web bolts - edge distance **LRFD [6.13.2.6.6]**:

Referring to Figure E24-2.7-1, it is clear that the minimum edge distance specified for this example is 1 1/2" and thus satisfies the minimum requirement **LRFD [Table 6.13.2.6.6-1]**.

The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or five inches.

$t := t_{wp}$ $\boxed{t_{wp} = 0.3750}$ in

The maximum edge distance allowable is:

$8 \cdot t_{wp} = 3.00$

The maximum distance from the corner bolts to the corner of the splice plate or girder flange is equal to (reference Figure E24-2.7-1):

$\boxed{\sqrt{1.50^2 + 1.50^2} = 2.12}$ in

and satisfies the maximum edge distance requirement.

$2.12 \cdot in \le 2.50 \cdot in$ OK

Web bolts - shear:

Calculate the polar moment of inertia, I_p, of the bolt group on each side of the centerline with respect to the centroid of the connection **LRFD [C6.13.6.1.4.b]**. This is required for determination of the shear force in a given bolt due to the applied web moments.

$$I_p := \frac{n \cdot m}{12}\left[s^2 \cdot \left(n^2 - 1\right) + g^2 \cdot \left(m^2 - 1\right)\right]$$

Where:

m = number of vertical rows of bolts

n = number of bolts in one vertical row

s = vertical pitch (in)

g = horizontal pitch (in)

$m := 2$

$n := 16$

$s := 3.00$ in

$g := 3.00$ in

The polar moment of inertia is:

$$I_p := \frac{n \cdot m}{12}\left[s^2 \cdot \left(n^2 - 1\right) + g^2 \cdot \left(m^2 - 1\right)\right] \qquad \boxed{I_p = 6192.00} \quad in^2$$

The total number of web bolts on each side of the splice, assuming two vertical rows per side with sixteen bolts per row, is:

$N_b := 32$

Strength I Limit State:

Under the most critical combination of the minimum design shear, moment and horizontal force, it is assumed that the bolts in the web splice have slipped and gone into bearing. The shear strength of an ASTM A325 7/8" diameter high-strength bolt in double shear, assuming the threads are excluded from the shear planes, was computed in E24-2.4 for Flange Bolts - Shear:

$R_u = 55.42$ kips

Case 1 - Dead load + positive live load:

The following forces were computed in E24-2.6:

$V_{uw} = 281.80$ kips

$M_{tot\_str\_pos} = 453.57$ kip-ft

$H_{uw\_str\_pos} = 515.37$ kips

The vertical shear force in the bolts due to the applied shear force:

$$P_{v\_str} := \frac{V_{uw}}{N_b} \qquad \boxed{P_{v\_str} = 8.81} \quad kips$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_str\_pos} := \frac{H_{uw\_str\_pos}}{N_b} \qquad \boxed{P_{H\_str\_pos} = 16.11} \quad kips$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

$$P_{Mv} := \frac{M_{total} \cdot x}{I_p}$$

and

$$P_{Mh} := \frac{M_{total} y}{I_p}$$

For the vertical component:

$$x := \frac{g}{2} \qquad \boxed{x = 1.50} \quad \text{in}$$

For the horizontal component:

$$y := \frac{15 \cdot s}{2} \qquad \boxed{y = 22.50} \quad \text{in}$$

Calculating the components:

$$P_{Mv\_str\_pos} := \frac{M_{tot\_str\_pos} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_str\_pos} = 1.32} \quad \text{kips}$$

$$P_{Mh\_str\_pos} := \frac{M_{tot\_str\_pos}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_str\_pos} = 19.78} \quad \text{kips}$$

The resultant bolt force for the extreme bolt is:

$$P_{r\_str\_pos} := \sqrt{\left(P_{v\_str} + P_{Mv\_str\_pos}\right)^2 + \left(P_{H\_str\_pos} + P_{Mh\_str\_pos}\right)^2} \qquad \boxed{P_{r\_str\_pos} = 37.28} \quad \text{kips}$$

Case 2 - Dead load + negative live load:

The calculations at the Strength I Limit State for case 2 are not shown since they are similar to those shown previously for case 1. The final check for case 2 is shown below.

The following forces were computed in 24E2.6:

$$V_{uw} = 281.80 \quad \text{kips}$$

$$M_{tot\_str\_neg} = 759.38 \quad \text{kip-ft}$$

$$H_{uw\_str\_neg} = -107.62 \quad \text{kip-ft}$$

The vertical shear force in the bolts due to the applied shear force:

$$P_{v\_str} := \frac{V_{uw}}{N_b} \qquad \boxed{P_{v\_str} = 8.81} \quad \text{kips}$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_str\_neg} := \frac{|H_{uw\_str\_neg}|}{N_b} \qquad \boxed{P_{H\_str\_neg} = 3.36} \quad \text{kips}$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

Calculating the components:

$$P_{Mv\_str\_neg} := \frac{M_{tot\_str\_neg} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_str\_neg} = 2.21} \quad \text{kips}$$

$$P_{Mh\_str\_neg} := \frac{M_{tot\_str\_neg}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_str\_neg} = 33.11} \quad \text{kips}$$

The resultant bolt force is:

$$P_{r\_str\_neg} := \sqrt{\left(P_{v\_str} + P_{Mv\_str\_neg}\right)^2 \cdots + \left(P_{H\_str\_neg} + P_{Mh\_str\_neg}\right)^2} \qquad \boxed{P_{r\_str\_neg} = 38.10} \quad \text{kips}$$

The governing resultant bolt force is:

$$P_{r\_str} := \max\left(P_{r\_str\_pos}, P_{r\_str\_neg}\right) \qquad \boxed{P_{r\_str} = 38.10} \quad \text{kips}$$

Check:

$P_{r\_str} = 38.10$ kips $<$ $R_u = 55.42$ kips OK

Service II Limit State:

The factored slip resistance, R_r, for a 7/8" diameter high-strength bolt in double shear for a Class B surface and standard holes was determined from E24-2.4 to be:

$R_r := 39.00$ kips

Case 1 - Dead load + positive live load:

The following forces were computed in E24-2.6:

$V_{w\_ser} = 195.50$ kips

$M_{tot\_ser\_pos} = 182.26$ kip-ft

$H_{uw\_ser\_pos} = 187.52$ kips

The vertical shear force in the bolts due to the applied shear force:

$$P_{s\_ser} := \frac{V_{w\_ser}}{N_b} \qquad \boxed{P_{s\_ser} = 6.11} \quad \text{kips}$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_ser\_pos} := \frac{H_{uw\_ser\_pos}}{N_b} \qquad \boxed{P_{H\_ser\_pos} = 5.86} \quad \text{kips}$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

For the vertical component:

$x = 1.50$ in

$$P_{Mv\_ser\_pos} := \frac{M_{tot\_ser\_pos} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_ser\_pos} = 0.53} \quad \text{kips}$$

For the horizontal component:

$y = 22.50$ in

$$P_{Mh\_ser\_pos} := \frac{M_{tot\_ser\_pos}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_ser\_pos} = 7.95} \quad \text{kips}$$

The resultant bolt force is:

$$P_{r\_ser\_pos} := \sqrt{\left(P_{s\_ser} + P_{Mv\_ser\_pos}\right)^2 + \left(P_{H\_ser\_pos} + P_{Mh\_ser\_pos}\right)^2} \qquad \boxed{P_{r\_ser\_pos} = 15.32} \quad \text{kips}$$

Case 2 - Dead load + negative live load:

The calculations at the Service II Limit State for case 2 are not shown since they are similar to those shown previously for case 1. The final check for case 2 is shown below.

The following forces were computed in 24E2.6:

$V_{w\_ser} = 195.50$ kips

$M_{tot\_ser\_neg} = 200.49$ kip-ft

$H_{uw\_ser\_neg} = -140.27$ kips

The vertical shear force in the bolts due to the applied shear force:

$$P_{s\_ser} := \frac{V_{w\_ser}}{N_b} \qquad \boxed{P_{s\_ser} = 6.11} \quad \text{kips}$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_ser\_neg} := \frac{\left|H_{uw\_ser\_neg}\right|}{N_b} \qquad \boxed{P_{H\_ser\_neg} = 4.38} \quad \text{kips}$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

For the vertical component:

$$P_{Mv\_ser\_neg} := \frac{M_{tot\_ser\_neg} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_ser\_neg} = 0.58} \text{ kips}$$

For the horizontal component:

$$P_{Mh\_ser\_neg} := \frac{M_{tot\_ser\_neg}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_ser\_neg} = 8.74} \text{ kips}$$

The resultant bolt force is:

$$P_{r\_ser\_neg} := \sqrt{\left(P_{s\_ser} + P_{Mv\_ser\_neg}\right)^2 \cdots + \left(P_{H\_ser\_neg} + P_{Mh\_ser\_neg}\right)^2} \qquad \boxed{P_{r\_ser\_neg} = 14.73} \text{ kips}$$

The governing resultant bolt force is:

$$P_{r\_ser} := \max\left(P_{r\_ser\_pos}, P_{r\_ser\_neg}\right) \qquad \boxed{P_{r\_ser} = 15.32} \text{ kips}$$

Check:

$$P_{r\_ser} = 15.32 \text{ kips} < R_r = 39.00 \text{ kips} \quad \text{OK}$$

Thirty-two 7/8" diameter high-strength bolts in two vertical rows on each side of the splice provides sufficient resistance against bolt shear and slip.

Shear yielding of splice plates **LRFD [6.13.6.1.4b]**:

Check for shear yielding on the gross section of the web splice plates under the Strength I design shear force, V_{uw}:

$$V_{uw} = 281.80 \text{ kips}$$

The factored resistance of the splice plates is taken as **LRFD [6.13.5.3]**:

$$R_r := \phi_v \cdot R_n$$

$$R_n := 0.58 \cdot A_g \cdot F_y$$

The gross area of the web splice is calculated as follows:

Number of splice plates:

$$N_{wp} := 2$$

Thickness of plate:

$t_{wp} := 0.375$ in

Depth of splice plate:

$d_{wp} := 48$ in

$A_{gross\_wp} := N_{wp} \cdot t_{wp} \cdot d_{wp}$ $\boxed{A_{gross\_wp} = 36.00}$ in^2

The factored shear resistance is then:

$\phi_v = 1.0$

$R_r := \phi_v \cdot (0.58) \cdot (A_{gross\_wp}) \cdot (F_y)$ $\boxed{R_r = 1044.00}$ kips

Check:

$V_{uw} = 281.80$ kips $<$ $R_r = 1044.00$ kips OK

Fracture and block shear rupture of the web splice plates **LRFD [6.13.6.1.4b]**:

Strength I Limit State checks for fracture on the net section of web splice plates and block shear rupture normally do not govern for plates of typical proportion. These checks are provided in this example for completeness.

From E24-2.6, the factored design shear for the Strength I Limit State was determined to be:

$V_{uw} = 281.80$ kips

Fracture on the net section **LRFD [C6.13.4]**:

Investigation of critical sections and failure modes, other than block shear, is recommended, including the case of a net section extending across the full plate width, and, therefore, having no parallel planes. This may be a more severe requirement for a girder flange or splice plate than the block shear rupture mode.

For this case, the areas of the plate resisting tension are considered to be zero.

$A_{tn} := 0.0$ in^2

Therefore, the factored resistance is:

$$R_r := \phi_{bs} \cdot (0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_y \cdot A_{tn})$$

Number of web plates:

$N_{wp} = 2$

Depth of the web plate:

$d_{wp} = 48.00$ in

Number of bolts along one plane:

$N_{fn} := 16$

Thickness of the web plate:

$t_{wp} = 0.3750$ in

Specified minimum tensile strength of the connected material:

$F_u := 65$ ksi

Diameter of the bolt holes:

$d_{hole} = 1.00$ in

Net area resisting shear:

$A_{vn} := N_{wp} \cdot (d_{wp} - N_{fn} \cdot d_{hole}) \cdot t_{wp}$ $\boxed{A_{vn} = 24.00}$ in^2

A_{vn} of the splice plates to be used in calculating the fracture strength of the splice plates cannot exceed eighty-five percent of the gross area of the plates **LRFD [6.13.5.2]**:

$A_{85} := 0.85 \cdot A_{gross\_wp}$ $\boxed{A_{gross\_wp} = 36.00}$ in^2

$A_{85} = 30.60$ in^2 > $A_{vn} = 24.00$ in^2 OK

The factored resistance is then:

$R_r := \phi_{bs} \cdot (0.58 \cdot F_u \cdot A_{vn})$ $\boxed{R_r = 723.84}$ kips

$R_r = 723.84$ kips > $V_{uw} = 281.80$ kips OK

Block shear rupture resistance **LRFD [6.13.4]**:

Connection plates, splice plates and gusset plates shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection.

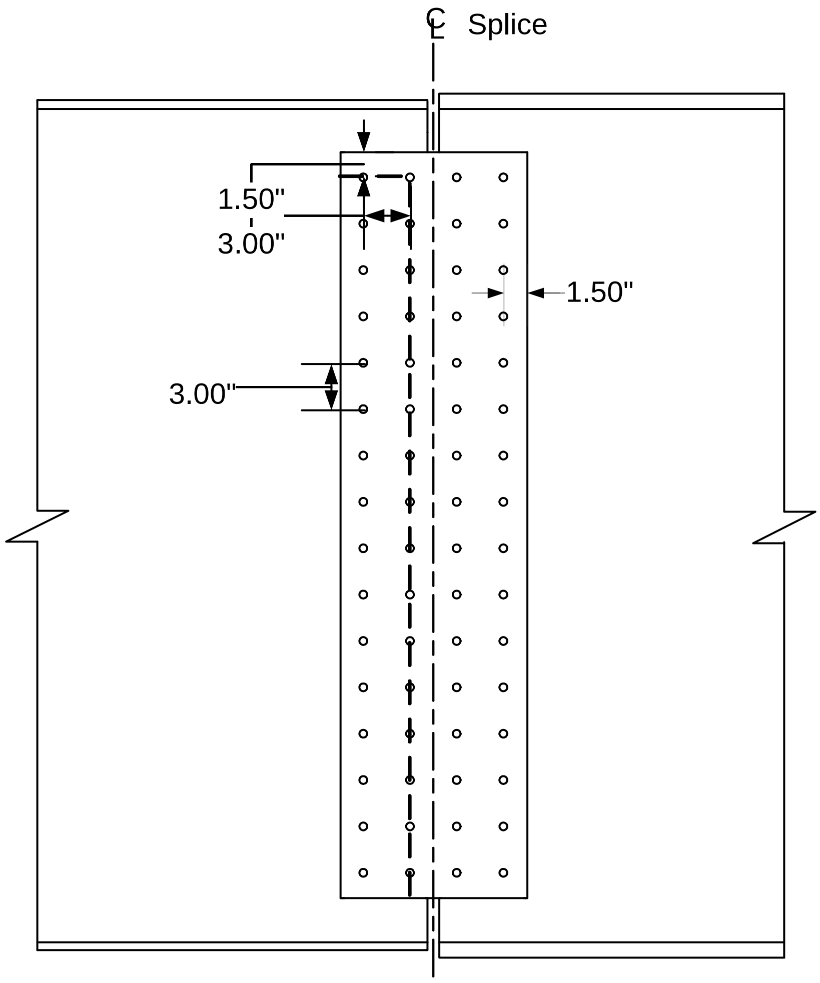

Figure E24-2.7-2
Block Shear Failure Mode - Web Splice Plate

Gross area along the plane resisting shear stress:

$A_{vg} := N_{wp} \cdot (d_{wp} - 1.50) \cdot t_{wp}$ $\boxed{A_{vg} = 34.88}$ in^2

Net area along the plane resisting shear stress:

$A_{vn} := N_{wp} \cdot [d_{wp} - 1.50 - 15.50 \cdot (d_{hole})] \cdot t_{wp}$ $\boxed{A_{vn} = 23.25}$ in^2

Net area along the plane resisting tension stress:

$A_{tn} := N_{wp} \cdot [1.50 + 3.0 - 1.5 \cdot (d_{hole})] \cdot t_{wp}$ $\boxed{A_{tn} = 2.25}$ in^2

$U_{bs} := 1.0$

$R_{r1} := \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_u \cdot A_{tn}) = 818.22$

$R_{r2} := \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_y \cdot A_{vg} + U_{bs} \cdot F_u \cdot A_{tn}) = 926.10$

$R_r := \min(R_{r1}, R_{r2})$

$\boxed{R_r = 818.22}$ kips

Check:

$V_{uw} = 281.80$ kips < $R_r = 818.22$ kips OK

Flexural yielding of splice plates **LRFD [6.13.6.1.4b]**:

Check for flexural yielding on the gross section of the web splice plates for the Strength I Limit State due to the total web moment and the horizontal force resultant:

$$f := \frac{M_{Total}}{S_{pl}} + \frac{H_{uw}}{A_{gross\_wp}} \le \phi_f \cdot F_y$$

$\phi_f = 1.0$ (see E24-2.1)

Section modulus of the web splice plate:

$$S_{pl} := \frac{1}{6} \cdot A_{gross\_wp} \cdot d_{wp}$$

$\boxed{S_{pl} = 288.00}$ in^3

Case 1 - Dead load + positive live load:

$M_{tot\_str\_pos} = 453.57$ kip-ft

$H_{uw\_str\_pos} = 515.37$ kips

$$f_{str\_pos} := \frac{M_{tot\_str\_pos}}{S_{pl}} \cdot (12) + \frac{H_{uw\_str\_pos}}{A_{gross\_wp}}$$

$\boxed{f_{str\_pos} = 33.21}$ ksi

$f_{str\_pos} = 33.21$ ksi < $\boxed{\phi_f \cdot F_y = 50.00}$ ksi OK

Case 2 - Dead load + negative live load:

$M_{tot\_str\_neg} = 759.38$ kip-ft

$H_{uw\_str\_neg} = -107.62$ kips

$$f_{str\_neg} := \frac{M_{tot\_str\_neg}}{S_{pl}} \cdot (12) + \frac{|H_{uw\_str\_neg}|}{A_{gross\_wp}}$$

$\boxed{f_{str\_neg} = 34.63}$ ksi

$f_{str\_neg} = 34.63$ ksi < $\boxed{\phi_f \cdot F_y = 50.00}$ ksi OK

Web bolts - bearing resistance at bolt holes **LRFD [6.13.2.9]**:

Since the girder web thickness is less than twice the thickness of the web splice plates, the girder web will control for the bearing check.

Check the bearing of the bolts on the connected material for the Strength I Limit State assuming the bolts have slipped and gone into bearing. The design bearing strength of the girder web at the location of the extreme bolt in the splice is computed as the minimum resistance along the two orthogonal shear failure planes shown in Figure E24-2.7-3. The maximum force (vector resultant) acting on the extreme bolt is compared to this calculated strength, which is conservative since the components of this force parallel to the failure surfaces are smaller than the maximum force.

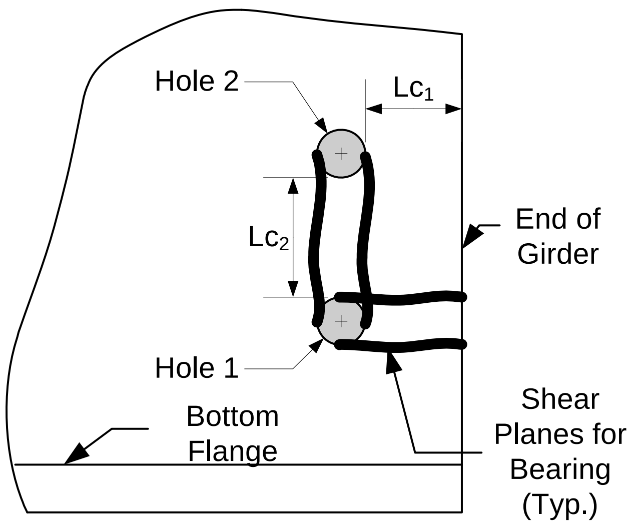

Figure E24-2.7-3
Bearing Resistance - Girder Web

To determine the applicable equation for the calculation of the nominal bearing resistance, the clear distance between holes and the clear end distance must be calculated and compared to the value of two times the nominal diameter of the bolt. This check yields:

$$\boxed{2 \cdot d_{bolt} = 1.75} \quad \text{in}$$

The edge distance from the center of the hole to the edge of the girder is taken as 1.75". Therefore, the clear distance between the edge of the hole and the edge of the girder is computed as follows **LRFD [6.13.2.6.6]**:

$$L_{c_1} := 1.75 - \frac{d_{hole}}{2} \qquad \boxed{L_{c_1} = 1.25} \quad \text{in}$$

The center-to-center distance between adjacent holes is 3". Therefore, the clear distance between holes is:

$$L_{c_2} := 3.00 - d_{hole} \qquad \boxed{L_{c_2} = 2.00} \quad \text{in}$$

For standard holes, where either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d **LRFD [6.13.2.9]**:

The nominal bearing resistance at the extreme bolt hole is as follows:

$R_n := 1.2 \cdot L_{c_1} \cdot t_w \cdot F_u$ $\boxed{R_n = 48.75}$ kips

The factored bearing resistance is:

$R_r := \phi_{bb} \cdot R_n$ $\boxed{R_r = 39.00}$ kips

The controlling minimum Strength I resultant bolt force was previously computed:

$P_{r\_str} = 38.10$ kips $<$ $R_r = 39.00$ kips OK

Fatigue of splice plates:

For load-induced fatigue considerations, each detail shall satisfy **LRFD [6.6.1.2.2]**:

$$\gamma \cdot (\Delta f) \le \Delta F_n$$

Fatigue is checked at the bottom edge of the splice plates, which by inspection are subject to a net tensile stress.

The normal stresses at the bottom edge of the splice plates due to the total positive and negative fatigue-load web moments and the corresponding horizontal force resultants are as follows:

$$f := \frac{M_{total}}{S_{pl}} + \frac{H_w}{A_{gross\_wp}}$$

From previous calculations:

$S_{pl} = 288.00$ in^3

$A_{gross\_wp} = 36.00$ in^3

Case 1 - Positive live load:

$M_{tot\_fat\_pos} = 54.48$ kip-ft (see E24-2.6)

$H_{uw\_fat\_pos} = 64.26$ kips (see E24-2.6)

$f_{fat\_pos} := \frac{M_{tot\_fat\_pos}}{S_{pl}} \cdot (12) + \frac{H_{uw\_fat\_pos}}{A_{gross\_wp}}$ $\boxed{f_{fat\_pos} = 4.05}$ ksi

Case 2 - Negative live load:

$M_{tot\_fat\_neg} = -54.39$ kip-ft (see E24-2.6)

$H_{uw\_fat\_neg} = -48.87$ kips (see E24-2.6)

$f_{fat\_neg} := \frac{M_{tot\_fat\_neg}}{S_{pl}} \cdot (12) + \frac{H_{uw\_fat\_neg}}{A_{gross\_wp}}$ $\boxed{f_{fat\_neg} = -3.62}$ ksi

The total fatigue-load stress range at the bottom edge of the web splice plates is therefore:

$\gamma\Delta f := |f_{fat\_pos}| + |f_{fat\_neg}|$ $\boxed{\gamma\Delta f = 7.68}$ ksi

From E24-2.4, the fatigue resistance was determined as:

$\Delta F_n = 16.00$ ksi

$\gamma\Delta f = 7.68$ ksi < $\Delta F_n = 16.00$ ksi OK

E24-2.8 Draw Schematic of Final Bolted Field Splice Design

Figure E24-2.8-1 shows the final bolted field splice as determined in this design example.

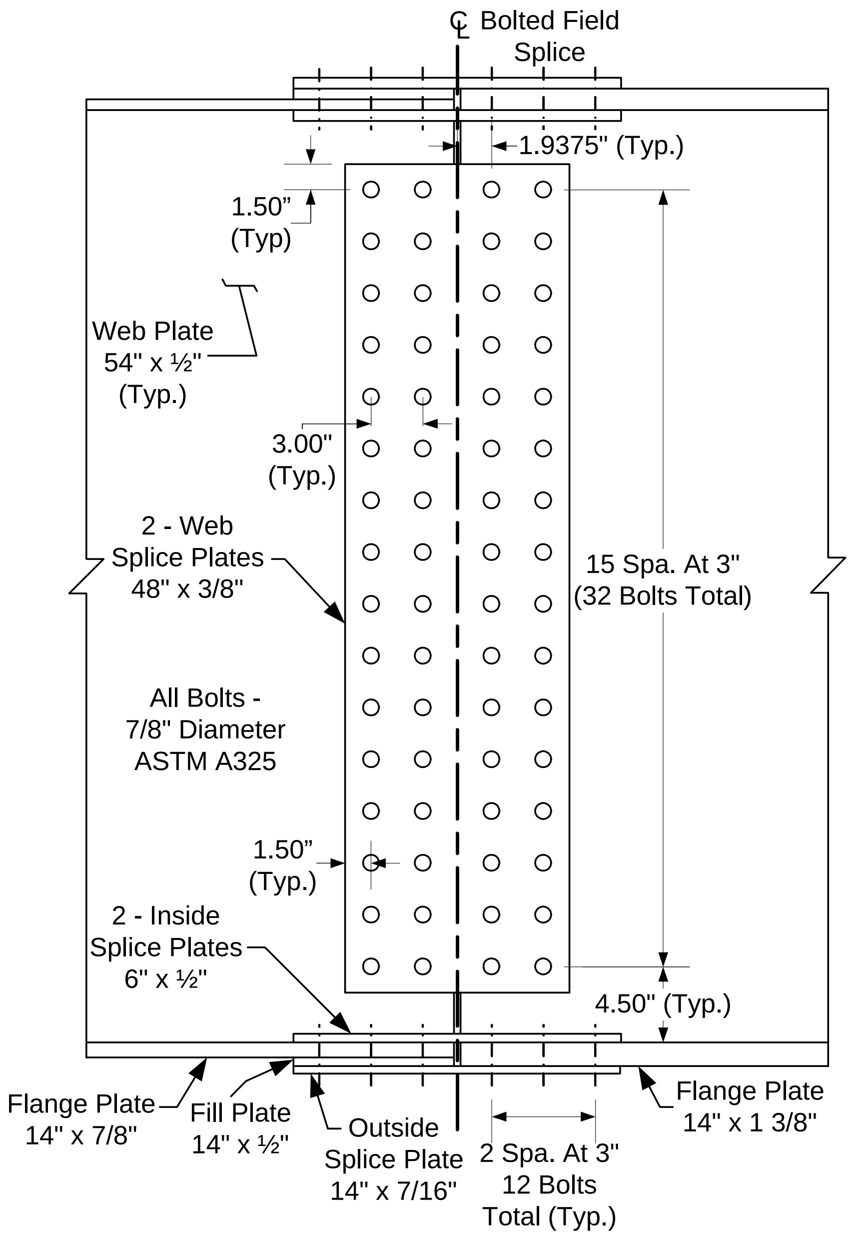

Figure E24-2.8-1
Final Bolted Field Splice Design

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Table of Contents

27.1 General

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports, ensuring that the bridge functions as intended. Bridges usually require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require greater consideration in minimizing forces caused by temperature change, friction and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

- Bridges are usually supported by reinforced concrete substructure units, and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as possible.

- Bridge bearings must be capable of withstanding and transferring dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

- Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance-free as possible.

WisDOT policy item:

The temperature range considered for steel girder superstructures is -30°F to 120°F. A temperature setting table for steel bearings is used for steel girders; where 45°F is the neutral temperature, resulting in a range of 120° - 45° = 75° for bearing design.

The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F. Using an installation temperature of 60° for prestressed girders, the resulting range is 60° - 5° = 55° for bearing design. For prestressed girders an additional shrinkage factor of 0.0003 ft/ft shall also be accounted for. No temperature setting table is used for prestressed concrete girders.

See the Standard for Steel Expansion Bearing Details to determine bearing plate “A” sizing (steel girders) or anchor plate sizing (prestressed concrete girders). This standard also gives an example of a temperature setting table for steel bearings when used for steel girders.

WisDOT policy item:

According to **LRFD [14.4.1]**, the influence of dynamic load allowance need not be included for bearings. However, dynamic load allowance shall be included when designing bearings for bridges in Wisconsin. Apply dynamic load allowance in **LRFD [3.6.2]** to HL-93 live loads as stated in **LRFD [3.6.1.2, 3.6.1.3]** and distribute these loads, along with dead loads, to the bearings.

27.2 Bearing Types

Bridge bearings are of two general types: expansion and fixed. Bearings can be fixed in both the longitudinal and transverse directions, fixed in one direction and expansion in the other, or expansion in both directions. Expansion bearings provide for rotational movements of the girders, as well as longitudinal movement for the expansion and contraction of the bridge spans. If an expansion bearing develops a large resistance to longitudinal movement due to corrosion or other causes, this frictional force opposes the natural expansion or contraction of the span, creating a force within the span that could lead to a maintenance problem in the future. Fixed bearings act as hinges by permitting rotational movement, while at the same time preventing longitudinal movement. The function of the fixed bearing is to prevent the superstructure from moving longitudinally off of the substructure units. Both expansion and fixed bearings transfer lateral forces, as described in **LRFD [Section 3]**, from the superstructure to the substructure units. Both bearing types are set parallel to the direction of structural movement; bearings are not set parallel to flared girders.

When deciding which bearings will be fixed and which will be expansion on a bridge, several guidelines are commonly considered:

- The bearing layout for a bridge must be developed as a consistent system. Vertical movements are resisted by all bearings, longitudinal horizontal movements are resisted by fixed bearings and facilitated in expansion bearings, and rotations are generally allowed to occur as freely as possible.

- For maintenance purposes, it is generally desirable to minimize the number of deck joints on a bridge, which can in turn affect the bearing layout.

- The bearing layout must facilitate the anticipated thermal movements, primarily in the longitudinal direction, but also in the transverse direction for wide bridges.

- It is generally desirable for the superstructure to expand in the uphill direction, wherever possible.

- If more than one substructure unit is fixed within a single superstructure unit, then forces will be induced into the fixed substructure units and must be considered during design. If only one pier is fixed, unbalanced friction forces from expansion bearings will induce force into the fixed pier.

- For curved bridges, the bearing layout can induce additional stresses into the superstructure, which must be considered during design.

- Forces are distributed to the bearings based on the superstructure analysis.

A valuable tool for selecting bearing types is presented in **LRFD [Table 14.6.2-1]**, in which the suitability of various bearing types is presented in terms of movement, rotation and resistance to loads. In general, it is best to use a fixed or semi-expansion bearing utilizing an unreinforced elastomeric bearing pad whenever possible, provided adverse effects such as excessive force transfer to the substructure does not occur. Where a fixed bearing is required with greater rotational capacity, steel fixed bearings can be utilized. Laminated elastomeric bearings are the preferred choice for expansion bearings. When such expansion bearings fail to meet project requirements, steel Type “A-T” expansion bearings should be used. For curved and/or highly skewed bridges, consideration should be given to the use of pot bearings.

27.2.1 Elastomeric Bearings

Elastomeric bearings are commonly used on small to moderate sized bridges. Elastomeric bearings are either fabricated as plain bearing pads (consisting of elastomer only) or as laminated (steel reinforced) bearings (consisting of alternate layers of steel reinforcement and elastomer bonded together during vulcanization). A sample plain elastomeric bearing pad is illustrated in Figure 27.2-1, and a sample laminated (steel reinforced) elastomeric bearing is illustrated in Figure 27.2-2.

These bearings are designed to transmit loads and accommodate movements between a bridge and its supporting structure. Plain elastomeric bearing pads can be used for small bridges, in which the vertical loads, translations and rotations are relatively small. Laminated (steel reinforced) elastomeric bearing pads are often used for larger bridges with more sizable vertical loads, translations and rotations. Performance information indicates that elastomeric bearings are functional and reliable when designed within the structural limits of the material. See **LRFD [Section 14]**, *AASHTO LRFD Bridge Construction Specifications*, Section 18, and *AASHTO M251* for design and construction requirements of elastomeric bearings.

WisDOT policy item:

WisDOT currently uses plain or laminated (steel reinforced) elastomeric bearings which are rectangular in shape. No other shapes or configurations are used for elastomeric bearings in Wisconsin.

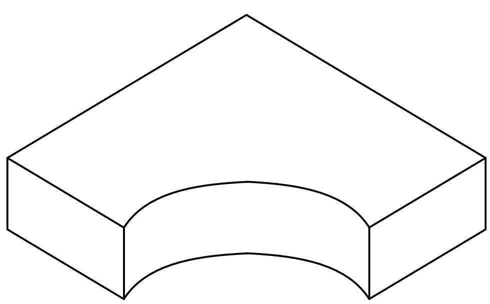

Figure 27.2-1
Plain Elastomeric Bearing

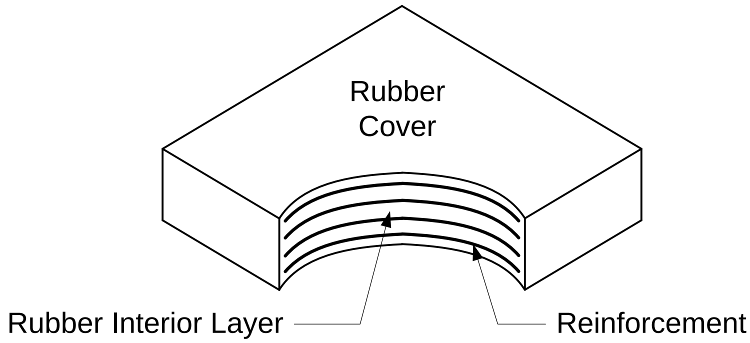

Figure 27.2-2
Laminated (Steel Reinforced) Elastomeric Bearing

AASHTO LRFD does not permit tapered elastomer layers in reinforced bearings. Laminated (steel reinforced) bearings must be placed on a level surface; otherwise gravity loads will produce shear strain in the bearing due to inclined forces. The angle between the alignment of the underside of the girder (due to the slope of the grade line, camber and dead load rotation) and a horizontal line must not exceed 0.01 radians, as per **LRFD [14.8.2]**. If the angle is greater than 0.01 radians or if the rotation multiplied by the top plate length is 1/8” or more, the 1 1/2" top steel plate must be tapered to provide a level load surface along the bottom of this plate under these conditions. The tapered plate will have a minimum thickness of 1 1/2" per *AASHTO Construction Specifications, Section 18*.

Plain and laminated (steel reinforced) elastomeric bearings can be designed by Method A as outlined in **LRFD [14.7.6]** and NCHRP-248 or by Method B as shown in **LRFD [14.7.5]** and NCHRP-298.

WisDOT policy item:

WisDOT uses Method A, as described in **LRFD [14.7.6]**, for elastomeric bearing design.

Method A results in a bearing with a lower capacity than a bearing designed using Method B. However, the increased capacity resulting from the use of Method B requires additional testing and quality control, and WisDOT currently does not have a system in place to verify these requirements.

For several years, plain elastomeric bearing pads have performed well on prestressed concrete girder structures. Refer to the Standard for Bearing Pad Details for Prestressed Concrete Girders for details. Prestressed concrete girders using this detail are fixed into the concrete diaphragms at the supports, and the girders are set on 1/2" thick plain elastomeric bearing pads. Laminated (steel reinforced) bearing details and steel plate and elastomer thicknesses are given on the Standard for Elastomeric Bearings for Prestressed Concrete Girders.

The design of an elastomeric bearing generally involves the following steps:

1. Obtain required design input **LRFD [14.4 & 14.6]**

The required design input for the design of an elastomeric bearing at the service limit state is dead load, live load plus dynamic load allowance, minimum vertical force due to permanent load, and design translation. The required design input at the strength limit state is shear force. Other required design input is expansion length, girder or beam bottom flange width, minimum grade of elastomer, and temperature zone. Two temperature zones are shown for Wisconsin in **LRFD [Figure 14.7.5.2-1]**, zones C and D. WisDOT policy is for all elastomeric bearings to meet Zone D requirements.

2. Select a feasible bearing type – plain or laminated (steel reinforced)

3. Select preliminary bearing properties **LRFD [14.7.6.2]**

   The preliminary bearing properties can be obtained from **LRFD [14.7.6.2]** or from past experience. The preliminary bearing properties include elastomer cover thickness, elastomer internal layer thickness, elastomer hardness, elastomer shear modulus, elastomer creep deflection, pad length, pad width, number of steel reinforcement layers, steel reinforcement thickness, steel reinforcement yield strength and steel reinforcement constant-amplitude fatigue threshold. WisDOT uses the following properties:

   - Elastomer cover thickness = 1/4"
   - Elastomer internal layer thickness = 1/2"
   - Elastomer hardness: Durometer 60 +/- 5
   - Elastomer shear modulus (G): 0.1125 ksi < G < 0.165 ksi
   - Elastomer creep deflection @ 25 years divided by instantaneous deflection = 0.30
   - Steel reinforcement thickness = 1/8"
   - Steel reinforcement yield strength = 36 ksi or 50 ksi
   - Steel reinforcement constant-amplitude fatigue threshold = 24 ksi

   However, not all of these properties are needed for a plain elastomeric bearing design.

4. Check shear deformation **LRFD [14.7.6.3.4]**

   Shear deformation, Δ_S, is the sum of deformation from thermal effects, Δ_{ST}, as well as creep and shrinkage effects, $\Delta_{Scr/sh}$ ($\Delta_S = \Delta_{ST} + \Delta_{Scr/sh}$).

   $$\Delta_{ST} = (\text{Expansion length})(\Delta_T)(\alpha)$$

   Where:

   Δ_T = Change in temperature (see 27.1) (degrees)

α	=	Coefficient of thermal expansion
	=	6×10^{-6} / °F for concrete, 6.5×10^{-6} / °F for steel

Shear deformation due to creep and shrinkage effects, $\Delta_{Scr/sh}$, should be added to Δ_{ST} for prestressed concrete girder structures. The value of $\Delta_{Scr/sh}$ is computed as follows:

$$\Delta_{Scr/sh} = (\text{Expansion length})(0.0003\ \text{ft/ft})$$

LRFD [14.7.6.3.4] provides shear deformation limits to help prevent rollover at the edges and delamination. The shear deformation, Δ_S, can be checked as specified in **LRFD [14.7.6.3.4]** and by the following equation:

$$h_{rt} \geq 2\,\Delta_S$$

Where:

h_{rt}	=	Smaller of total elastomer or bearing thickness (inches)
Δ_S	=	Maximum total shear deformation of the bearing at the service limit state (inches)

5. Check compressive stress **LRFD [14.7.6.3.2]**

The compressive stress, σ_S, at the service limit state can be checked as specified in **LRFD [14.7.6.3.2]** and by the following equations:

$\sigma_s \leq 0.80\ ksi$ and $\sigma_s \leq 1.00GS$ for plain elastomeric pads

$\sigma_s \leq 1.25\ ksi$ and $\sigma_s \leq 1.25GS$ for laminated (steel reinforced) elastomeric pads

Where:

σ_s	=	Service average compressive stress due to total load (ksi)
G	=	Shear modulus of elastomer (ksi)
S	=	Shape factor for the thickest layer of the bearing

LRFD [14.7.6.3.2] states that the stress limits may be increased by 10 percent where shear deformation is prevented, but this is not considered applicable to WisDOT bearings.

The shape factor for individual elastomer layers is the plan area divided by the area of the perimeter free to bulge. For laminated (steel reinforced) elastomeric bearings, the following requirements must be satisfied before calculating the shape factor:

- All internal layers of elastomer must be the same thickness.
- The thickness of the cover layers cannot exceed 70 percent of the thickness of the internal layers.

The shape factor, S_i, for rectangular bearings without holes can be determined as specified in **LRFD [14.7.5.1]** and by the following equation:

$$S_i = \frac{LW}{2h_{ri}(L+W)}$$

Where:

S_i	=	Shape factor for the i^{th} layer
h_{ri}	=	Thickness of i^{th} elastomeric layer in elastomeric bearing (inches)
L	=	Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)
W	=	Width of the bearing in the transverse direction (inches)

6. Check stability **LRFD [14.7.6.3.6]**

For stability, the total thickness of the rectangular pad must not exceed one-third of the pad length or one-third of the pad width as specified in **LRFD [14.7.6.3.6]**, or expressed mathematically:

$$H \le \frac{L}{3} \text{ and } H \le \frac{W}{3}$$

Where:

H	=	Total thickness of the elastomeric bearing (excluding top plate) (inches)
L	=	Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)
W	=	Width of the bearing in the transverse direction (inches)

7. Check compressive deflection **LRFD [14.7.5.3.6, 14.7.6.3.3]**

The compressive deflection, δ, of the bearing shall be limited to ensure the serviceability of the deck joints, seals and other components of the bridge. Deflections of elastomeric bearings due to total load and to live load alone should be considered separately. Relative deflections across joints must be restricted so that a step doesn't occur at a deck joint. **LRFD [C14.7.5.3.6]** recommends that a maximum relative live load deflection across a joint be limited to 1/8".

WisDOT policy item:

WisDOT uses a live load + creep deflection limit of 1/8" for elastomeric bearing design.

Laminated (steel reinforced) elastomeric bearings have a nonlinear load deflection curve in compression. In the absence of information specific to the particular elastomer to be used, **LRFD [Figure C14.7.6.3.3-1]** may be used as a guide. Creep effects should be determined from information specific to the elastomeric compound used. Use the material properties given in this section. The compressive deflection, δ, can be determined as specified in **LRFD [14.7.5.3.6, 14.7.6.3.3]** and by the following equation:

$$\delta = \sum \varepsilon_i \, h_{ri}$$

Where:

δ = Instantaneous deflection (inches)

ε_i = Instantaneous compressive strain in the i^{th} elastomer layer of a laminated (steel reinforced) bearing

h_{ri} = Thickness of i^{th} elastomeric layer in a laminated (steel reinforced) bearing (inches)

Based on **LRFD [14.7.6.3.3]**, the initial compressive deflection of a plain elastomeric pad or in any layer of a laminated (steel reinforced) elastomeric bearing at the service limit state without dynamic load allowance shall not exceed $0.09h_{ri}$.

8. Check anchorage

WisDOT exception to AASHTO:

Design anchorage for laminated elastomeric bearings if the unfactored dead load stress is less than 200 psi. This is an exception to **LRFD [14.8.3]** based on past practice and good performance of existing bearings.

The factored force due to the deformation of an elastomeric element shall be taken as specified in **LRFD [14.6.3.1]** by the following equation:

$$H_u > GA \frac{\Delta_u}{h_{rt}}$$

Where:

H_u = Lateral load from applicable strength load combinations in **LRFD [Table 3.4.1-1]** (kips)

G = Shear modulus of the elastomer (ksi)

A = Plan area of elastomeric element or bearing ($inches^2$)

Δ_u = Factored shear deformation (inches)

h_{rt} = Total elastomer thickness (inches)

9. Check reinforcement **LRFD [14.7.5.3.5, 14.7.6.3.7]**

Reinforcing steel plates increase compressive and rotational stiffness, while maintaining flexibility in shear. The reinforcement must have adequate capacity to handle the tensile stresses produced in the plates as they counter the lateral bulging of the elastomer layers due to compression. These tensile stresses increase with compressive load. The reinforcement thickness must also satisfy the requirements of the *AASHTO LRFD Bridge Construction Specifications*. The reinforcing steel plates can be checked as specified in **LRFD [Equation 14.7.5.3.5-1,2]**:

$$h_s \geq \frac{3\,h_{max}\,\sigma_s}{F_y} \text{ for service limit state}$$

$$h_s \geq \frac{2.0\,h_{max}\,\sigma_L}{\Delta F_{TH}} \text{ for fatigue limit state}$$

Where:

h_s = Thickness of the steel reinforcement (inches)

h_{max} = Thickness of the thickest elastomeric layer in elastomeric bearing (inches)

σ_s = Service average compressive stress due to total load (ksi)

Fy = Yield strength of steel reinforcement (ksi)

σ_L = Service average compressive stress due to live load (ksi)

ΔF_{TH} = Constant amplitude fatigue threshold for Category A as specified in **LRFD [6.6]** (ksi)

If holes exist in the reinforcement, the minimum thickness shall be increased by a factor equal to twice the gross width divided by the net width.

10. Rotation **LRFD [14.7.6.3.5]**

WisDOT exception to AASHTO:

Lateral rotation about the longitudinal axis of the bearing shall not be considered for straight girders.

WisDOT policy item:

Per **LRFD [14.8.2]**, a tapered plate shall be used if the inclination of the underside of the girder to the horizontal exceeds 0.01 radians. Additionally, if the rotation multiplied by the plate length is 1/8 inch or more, taper the plate.

27.2.2 Steel Bearings

For fixed bearings, a rocker plate attached to the girder is set on a masonry plate which transfers the girder reaction to the substructure unit. The masonry plate is attached to the substructure unit with anchor bolts. Pintles set into the masonry plate prevent the rocker from sliding off the masonry plate while allowing rotation to occur. This bearing is represented on the Standard for Fixed Bearing Details Type "A" - Steel Girders.

For expansion bearings, two additional plates are utilized, a stainless steel top plate and a Teflon plate allowing expansion and contraction to occur, but not in the transverse direction. This bearing is shown on the Standard for Stainless Steel - TFE Expansion Bearing Details Type "A-T".

Type "B" rocker bearings have been used for reactions greater than 400 kips and having a requirement for smaller longitudinal forces on the substructure unit. However, in the future, WisDOT plans to eliminate rocker bearings for new bridges and utilize pot bearings.

Pot and disc bearings are commonly used for moderate to large bridges. They are generally used for applications requiring a multi-directional rotational capacity and a medium to large range of load.

Hold down devices are additional details added to the Type "A-T" bearings for situations where live load can cause uplift at the abutment end of a girder. Ideally, proper span configurations would eliminate the need for hold down devices as they have proven to be a maintenance problem.

Since strength is not the governing criteria, anchor bolts are designed with Grade 36 steel for all steel bearings.

27.2.2.1 Type "A" Fixed Bearings

Type "A" Fixed Bearings prevent translation both transversely and longitudinally while allowing rotation in the longitudinal direction. This bearing is represented on the Standard for Fixed Bearing Details Type "A" - Steel Girders. An advantage of this bearing type is that it is very low maintenance. See 27.2.2.2 Type "A-T" Expansion Bearings for design information.

27.2.2.2 Type "A-T" Expansion Bearings

Type "A-T" Expansion bearings are designed to translate by sliding an unfilled polytetrafluoroethylene (PTFE or TFE) surface across a smooth, hard mating surface of stainless steel. Expansion bearings of Teflon are not used without provision for rotation. A rocker plate is provided to facilitate rotation due to live load deflection or change of camber. The Teflon sliding surface is bonded to a rigid back-up material capable of resisting horizontal shear and bending stresses to which the sliding surfaces may be subjected.

Design requirements for TFE bearing surfaces are given in **LRFD [14.7.2]**. Stainless steel-TFE expansion bearing details are given on the Standard for Stainless Steel – TFE Expansion Bearing Details Type "A-T."

Friction values are given in the **LRFD [14.7.2.5]**; they vary with loading and temperature. It is permissible to use 0.10 for a maximum friction value and 0.06 for a minimum value when determining unbalanced friction forces.

The design of type "A-T" bearings is relatively simple. The first consideration is the rocker plate length which is proportional to the contact stress based on a radius of 24" using Grade 50W steel. The rocker plate thickness is determined from a minimum of 1 1/2" to a maximum computed from the moment by assuming one-half the bearing reaction value (N/2) acting at a lever arm of one-fourth the width of the Teflon coated plate (W/4) over the length of the rocker plate. The Teflon coated plate is designed with a minimum width of 7" and the allowable stress as specified in **LRFD [14.7.2.4]** on the gross area; in many cases this controls the capacity of the expansion bearings as given in the Standard for Stainless Steel – TFE Expansion Bearing Details Type "A-T."

The design of the masonry plate is based on a maximum allowable bearing stress as specified in **LRFD [14.8.1]**. The masonry plate thickness is determined from the maximum bending moments about the x-or y-axis using a uniform pressure distribution.

In lieu of designing specific bearings, the designer may use Service I limit state loading, including dynamic load allowance, and Standards for Fixed Bearing Details Type “A” – Steel Girders, Stainless Steel – TFE Expansion Bearing Details Type “A-T” and Steel Bearings for Prestressed Concrete Girders to select the appropriate bearing.

27.2.2.3 High-Load Multi-Rotational Bearings

High-Load Multi-Rotational bearings, such as pot or disc bearings, are commonly used for moderate to large bridges. They are generally used for curved and/or highly skewed bridge applications requiring a multi-directional rotational capacity and a medium to large range of load.

Pot bearings consist of a circular non-reinforced neoprene or rubber pad, of relatively thin section, which is totally enclosed by a steel pot. The rubber is prevented from bulging by the pot containing it and acts similar to a fluid under high pressure. The result is a bearing providing suitable rotation and at the same time giving the effect of a point-contact rocker bearing since the center of pressure does not vary more than 4 percent. As specified in **LRFD [14.7.4.1]**, the minimum vertical load on a pot bearing should not be less than 20 percent of the vertical design load.

Pot bearings resist vertical load primarily through compressive stress in the elastomeric pad. The pad can deform and it has some shear stiffness, but it has very limited compressibility. Pot bearings generally have a large reserve of strength against vertical load. Pot bearings facilitate rotation through deformation of the elastomeric pad. During rotation, one side of the pad compresses and the other side expands. Pot bearings can sustain many cycles of small rotations with little or no damage. However, they can experience significant damage when subjected to relatively few cycles of large rotations.

Pot bearings can also resist horizontal loads. They can either be fixed, guided or non-guided. Fixed pot bearings (see Figure 27.2-3) can not translate in any direction, and they resist horizontal load primarily through contact between the rim of the piston and the wall of the pot. Guided pot bearings (see Figure 27.2-4) can translate in only one direction, and they resist horizontal load in the other direction through the use of guide bars. Non-guided pot bearings (see Figure 27.2-5) can translate in any direction, and they do not resist horizontal loads in any direction.

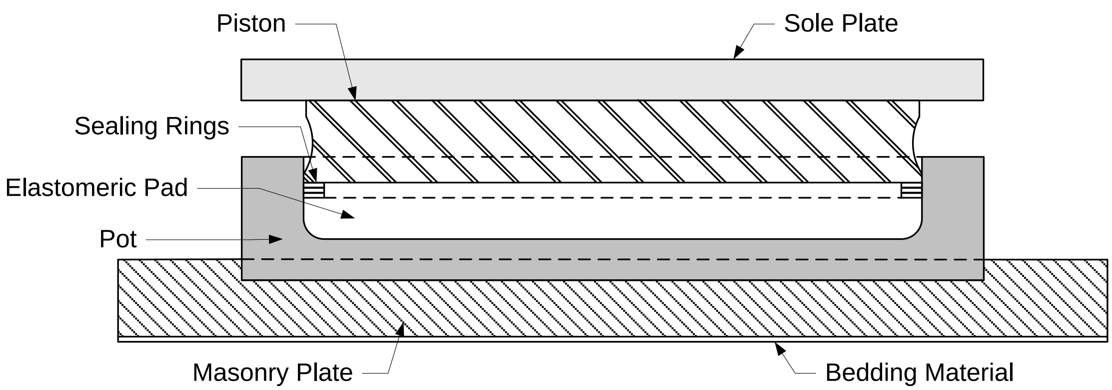

Figure 27.2-3
Fixed Pot Bearing

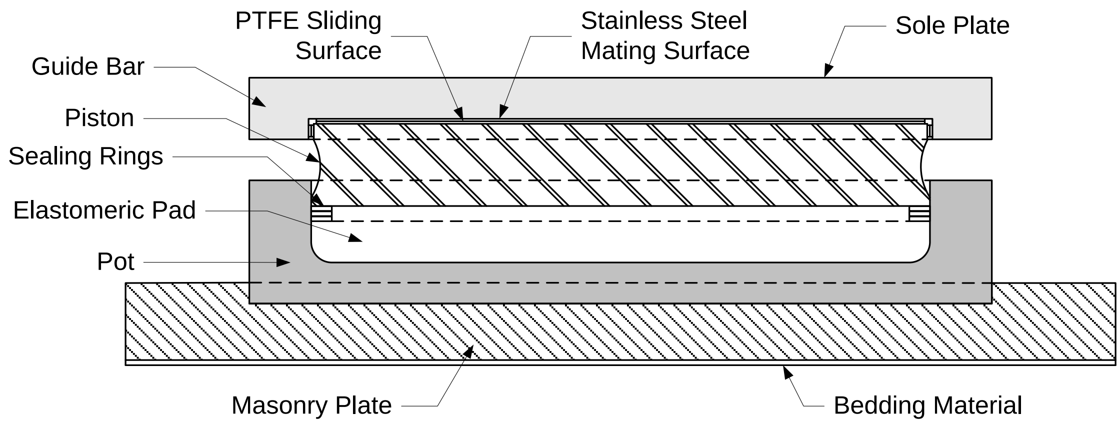

Figure 27.2-4
Guided Pot Bearing

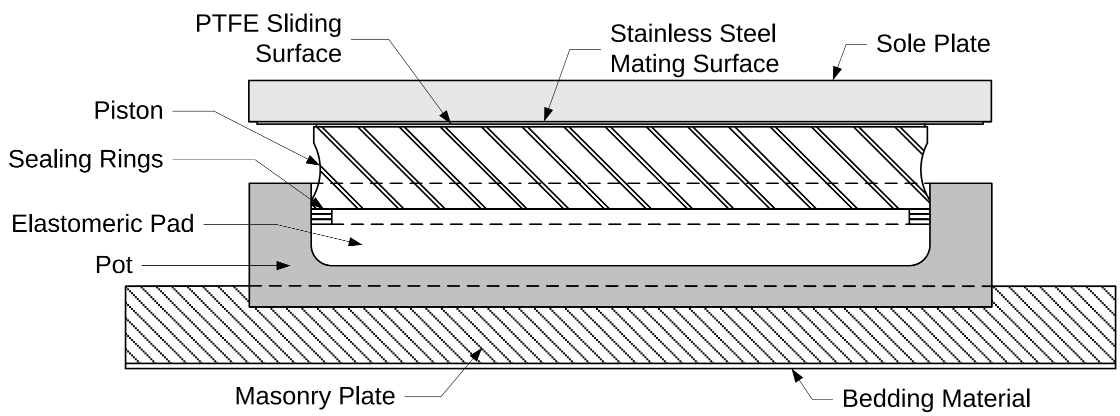

Figure 27.2-5
Non-Guided Pot Bearing

Disc bearings consist of a circular polyether urethane disc, confined by upper and lower steel plates and held in place by a positive location device. Limiting rings, either steel rings welded to the upper and lower steel plates or a circular recess in each of those plates, may also be used to partially confine the elastomer against lateral expansion. A shear-resisting mechanism shall be provided and it may be placed either inside or outside of the polyether urethane disc.

Disc bearings function by deformation of the polyether urethane disc, which should be stiff enough to resist vertical loads without excessive deformation and yet be flexible enough to accommodate the imposed rotations without liftoff or excessive stress on other components of the bearing assembly. The shear-resisting mechanism transmits horizontal forces between the upper and lower steel plates. As specified in **LRFD [14.7.8.4]**, the shear-resisting mechanism shall be capable of resisting a horizontal force in any direction equal to the larger of the design shear force at the strength and extreme event limit states or 15 percent of the design vertical load at the service limit state.

Disc bearings can either be fixed, guided or non-guided. Fixed disc bearings (see Figure 27.2-6) cannot translate in any direction. Guided disc bearings (see Figure 27.2-7) can translate in only one direction. Non-guided disc bearings (see Figure 27.2-8) can translate in any direction.

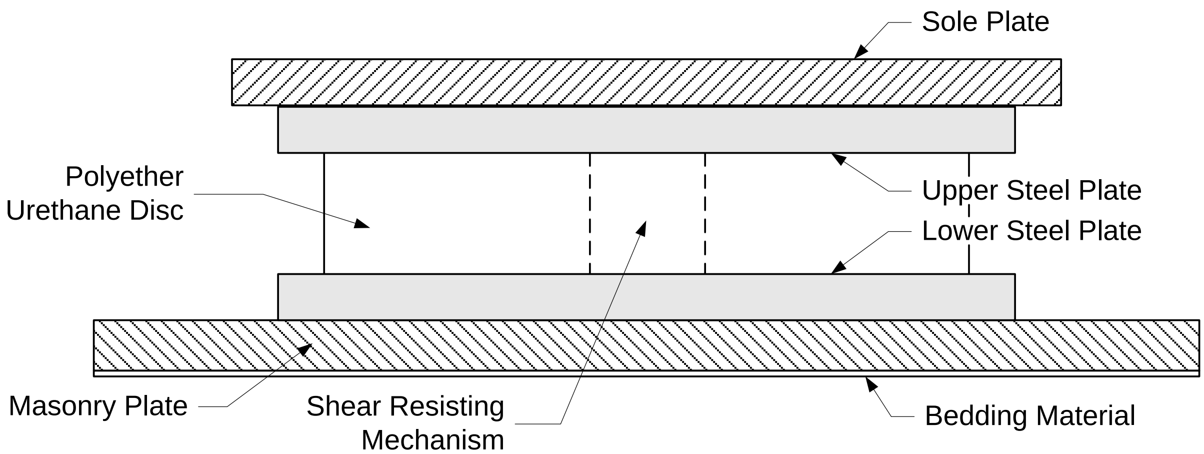

Figure 27.2-6
Fixed Disc Bearing

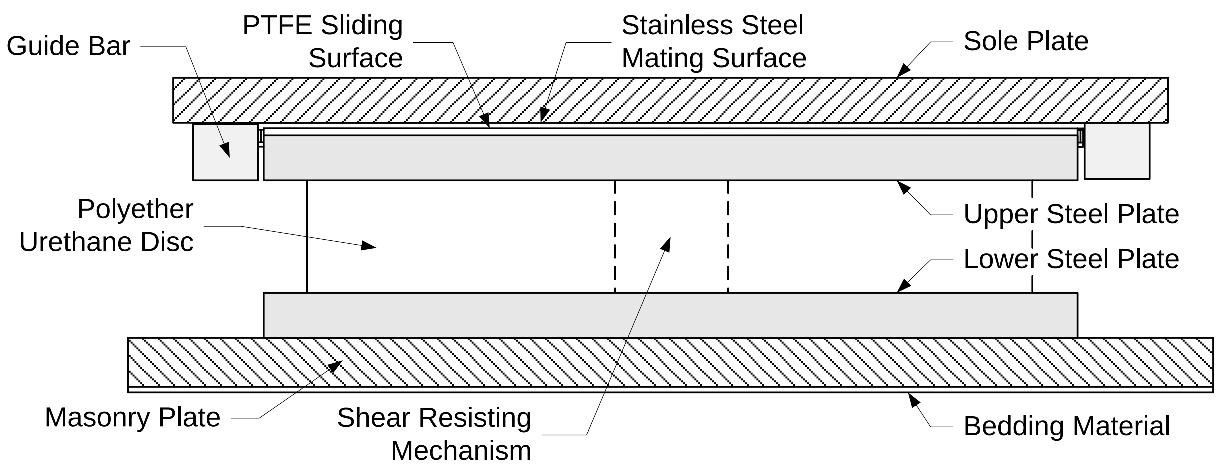

Figure 27.2-7
Guided Disc Bearing

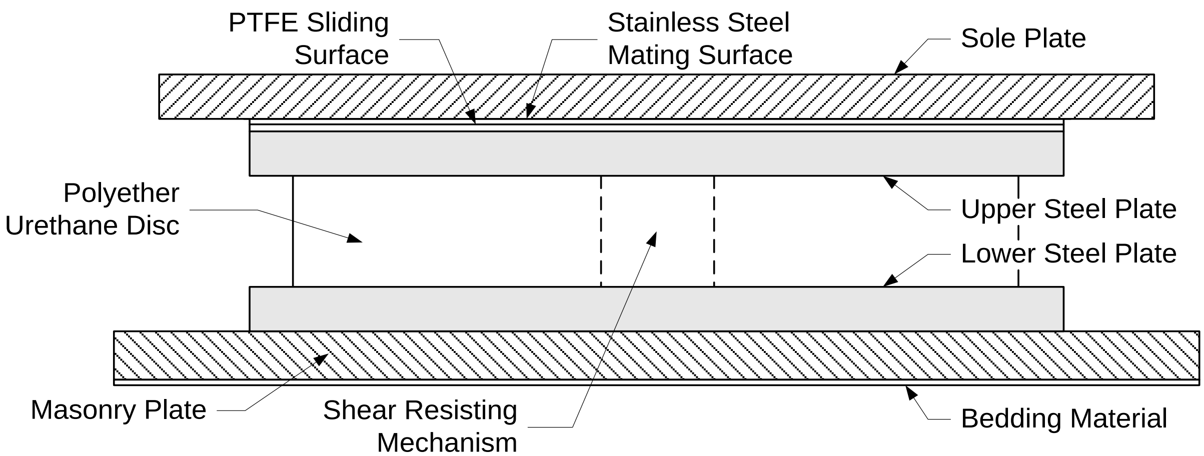

Figure 27.2-8
Non-Guided Disc Bearing

The design of a pot or disc bearing generally involves the following steps:

1. Obtain required design input **LRFD [14.4 & 14.6]**
2. Select a feasible bearing type: fixed, guided or non-guided
3. Select preliminary bearing properties
    a. Pot bearings: **LRFD [14.7.4.2]**
    b. Disc bearings: **LRFD [14.7.8.2]**
4. Design the bearing elements
    a. Pot bearings:
        i. Design the elastomeric disc **LRFD [14.7.4.3 and 14.7.4.4]**
        ii. Design the sealing rings **LRFD [14.7.4.5]**
        iii. Design the pot **LRFD [C14.7.4.3, 14.7.4.6 and 14.7.4.7]**
        iv. Design the piston **LRFD [14.7.4.7]**
    b. Disc bearings:
        i. Design the elastomeric disc **LRFD [14.7.8.3]**
        ii. Design the shear resisting mechanism **LRFD [14.7.8.4]**

5. Design the guides and restraints, if applicable **LRFD [14.7.9]**

6. Design the PTFE sliding surface, if applicable **LRFD [14.7.2]**

7. Design the sole plate, masonry plate (or bearing plate), anchorage and connections for pot bearings; design the sole plate, masonry plate (or bearing plate), upper and lower plates, anchorage and connections for disc bearings; as applicable **LRFD [6, 14.8 and 14.7.8.5]**

8. Check the concrete or steel support **LRFD [5.7.5 and 6]**

Although the steps for pot and/or disc bearing design are given above, the actual bearing design is typically done by the manufacturer. The design of the masonry plate is done either by the design engineer or by the bearing manufacturer (this should be coordinated and noted in the contract documents).

When using pot or disc bearings, the design plans need to specify the following:

- Degree of fixity (fixed, guided in one direction or non-guided)
- Maximum vertical load
- Minimum vertical load
- Maximum horizontal load (fixed and guided, only)
- Assumed bearing height

Note: The loads specified shall be Service I limit state loads, including dynamic load allowance.

Field adjustments to the given beam seat elevations will be required if the actual bearing height differs from the assumed bearing height stated on the plan. To facilitate such an adjustment without affecting the structural integrity of the substructure unit, a concrete pedestal (plinth) is detailed at each bearing location. Detailing a pedestal height of 10” based on the assumed bearing height will give sufficient room for adjustment should the actual bearing height differ from the assumed bearing height.

27.3 Hold Down Devices

Hold down devices are additional elements added to the Type "A-T" bearings for situations where live load can cause uplift at the abutment end of a girder. Ideally, proper span configurations would eliminate the need for hold down devices as they have proven to be a maintenance problem. Details for hold down devices are given in the Standard for Hold Down Devices.

27.4 Design Example

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E27-1 DESIGN EXAMPLE - STEEL REINFORCED ELASTOMERIC BEARING

ThIs design example is for a 3-span prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.
(Example is current through LRFD Seventh Edition - 2016 Interim)

E27-1.1 Design Data

Bearing location: Abutment (Type A3)

Girder type: 72W

$L_{exp} := 220$ Expansion length, ft

$b_f := 2.5$ Bottom flange width, ft

$DL_{serv} := 167$ Service I limit state dead load, kips

$DL_{ws} := 23$ Service I limit state future wearing surface dead load, kips

$LL_{serv} := 62$ Service I limit state live load, kips

$h_{rcover} := 0.25$ Elastomer cover thickness, in

$h_s := 0.125$ Steel reinforcement thickness, in

$F_y := 36$ Minimum yield strength of the steel reinforcement, ksi

Temperature Zone:	D (Use for Entire State)	**LRFD [Fig. 14.7.5.2-1]**
Minimum Grade of Elastomer:	4	**LRFD [Table 14.7.5.2-1]**
Elastic Hardness:	Durometer 60 +/- 5	(used 55 for design)
Shear Modulus (G):	0.1125 ksi < G <0.165 ksi	**LRFD [Table 14.7.6.2-1]**
Creep Deflection @ 25 Years		
divided by instantaneous deflection:	0.3	**LRFD [Table 14.7.6.2-1]**

E27-1.2 Design Method

Use Design Method A **LRFD [14.7.6]**
Method A results in a bearing with a lower capacity than a bearing designed using Method B. However the increased capacity resulting from the use of Method B requires additional testing and quality control.

E27-1.3 Dynamic Load Allowance

The influence of impact need not be included for bearings **LRFD [14.4.1]**; however, dynamic load allowance will be included to follow a **WisDOT policy item**.

E27-1.4 Shear

The maximum shear deformation of the pad shall be taken as the maximum horizontal superstructure displacement, reduced to account for the pier flexibility. **LRFD [14.7.6.3.4]**

$h_{rt} \geq 2\Delta_s$ **LRFD [Equation 14.7.6.3.4-1]**

Temperature range: T_{low} and T_{high} values below are from WisDOT policy item in 27.1

$T_{low} := 5$ Minimum temperature, °F

$T_{high} := 85$ Maximum temperature, °F

$\gamma_{TU} := 1.2$ Service I Load factor for deformation **LRFD [Table 3.4.1-1]**

$T_{install} := 60$ Installation temperature, °F

$\alpha_c := 0.000006$ Coefficient of thermal expansion of concrete, ft/ft/°F

$S_{crsh} := 0.0003$ Coefficient of creep and shrinkage of concrete, ft/ft

$\Delta_T := T_{install} - T_{low}$ $\boxed{\Delta_T = 55}$ °F

Maximum total shear deformation of the elastomer

$\Delta_s := L_{exp} \cdot \alpha_c \cdot \Delta_T \cdot 12 + L_{exp} \cdot S_{crsh} \cdot 12$ $\boxed{\Delta_s = 1.663}$ in

Required total elastomer thickness

$H_{rt} \geq 2 \cdot \gamma_{TU} \cdot \Delta_s$ $\boxed{H_{rt} = 3.992}$ in

Elastomer internal layer thickness

$h_{ri} := 0.5$ in

Required elastomer thickness **LRFD [14.7.6.1)**

$\frac{h_{rcover}}{h_{ri}} \leq 0.7$ $\boxed{\frac{h_{rcover}}{h_{ri}} = 0.5}$

$\boxed{check = \text{"< 0.7, OK"}}$

Determine the number of internal elastomer layers:

$n := \frac{H_{rt} - 2 \cdot h_{rcover}}{h_{ri}}$ Note: $\boxed{h_{rcover} = 0.25}$ in

$\boxed{n = 6.983}$ layers

Use: $\boxed{n = 7}$ layers

Total elastomer thickness:

$h_{rt} := 2 \cdot h_{rcover} + n \cdot h_{ri}$ $\boxed{h_{rt} = 4.0}$ in

Total height of reinforced elastomeric pad:

$H := h_{rt} + (n + 1) \cdot h_s$ $\boxed{H = 5.000}$ in

E27-1.5 Compressive Stress

$\sigma_{s\_all} \le 1.25$ and $\sigma_{s\_all} \le 1.25 \cdot G \cdot S$ **LRFD [14.7.6.3.2]**

$edge := 3$ in Transverse distance from the edge of the flange to edge of bearing

$W := 12 \cdot b_f - 2 \cdot edge$ Transverse dimension $\boxed{W = 24}$ in

$L \ge \dfrac{DL_{serv} + LL_{serv}}{W \cdot \sigma_{s\_all}}$ Since $\sigma_{s\_all} \le \dfrac{DL_{serv} + LL_{serv}}{L \cdot W}$

$\sigma_{s\_all} := 1.25$ ksi

$L := \dfrac{DL_{serv} + LL_{serv}}{W \cdot \sigma_{s\_all}}$ Longitudinal dimension $\boxed{L = 7.633}$ in

$increment := 5$ in <== Rounding increment

$\boxed{L = 10}$ in

(Use a 1 inch minimum rounding increment for design. For this example, the rounding increment is used to increase L dimension to satisfy subsequent stress checks, etc.)

Determine shape factor for internal layer **LRFD [Equation 14.7.5.1-1]**

$S_i := \dfrac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ $\boxed{S_i = 7.059}$

$G := 0.1125$ ksi $0.1125\text{ksi} < G < 0.165\text{ksi}$

$\boxed{1.25 \cdot G \cdot S_i = 0.993}$ ksi (Verify that **LRFD** is satisfied for a full range of G values. The minimum G values is used here. See also E27-1.8)

$\sigma_s := \dfrac{DL_{serv} + LL_{serv}}{L \cdot W}$ $\boxed{\sigma_s = 0.954}$ ksi

$\boxed{\sigma_s = \text{"< 1.25GS, OK"}}$

E27-1.6 Stability

$H \leq \frac{L}{3}$ and $H \leq \frac{W}{3}$ **LRFD [14.7.6.3.6]**

$\boxed{H = 5.000}$ in

Bearing length check:

$L_{min} := 3 \cdot H$ $\boxed{L_{min} = 15}$ in

$\boxed{L = 10}$ in

Use the larger value: $\boxed{L = 15}$ in

Bearing width check:

$W_{min} := 3 \cdot H$ $\boxed{W_{min} = 15}$ in

$\boxed{W = 24}$ in

Use the larger value: $\boxed{W = 24}$ in

Revised shape factor and compressive stress for internal layer:

$\boxed{h_{ri} = 0.5}$ in

$\boxed{G = 0.1125}$ ksi

$S_i := \frac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ $\boxed{S_i = 9.231}$

$\boxed{1.25 \cdot G \cdot S_i = 1.298}$ ksi

$\sigma_s := \frac{DL_{serv} + LL_{serv}}{L \cdot W}$ $\boxed{\sigma_s = 0.636}$ ksi

$\boxed{\sigma_s = \text{"< 1.25GSi, OK"}}$

Check **LRFD [C14.7.6.1]**: $S_i^2 / n < 20$ (for rectangular shape with $n \geq 3$)

$S_i^2 / n = (9.231)^2 / 8 = 10.7 < 20$ "OK"

where n = (7 inter. layers + 1/2 (2 exter. layers)) = 8

Revised shape factor and compressive stress for the cover layer:

$\boxed{h_{rcover} = 0.25}$ in

$$S_{cover} := \frac{L \cdot W}{2 \cdot h_{rcover} \cdot (L + W)}$$

$\boxed{S_{cover} = 18.462}$ ksi

$\boxed{1.25 \cdot G \cdot S_{cover} = 2.596}$ ksi

$$\sigma_S := \frac{DL_{serv} + LL_{serv}}{L \cdot W}$$

$\boxed{\sigma_S = 0.636}$ ksi

$\boxed{\sigma_S = \text{"< 1.25GS, OK"}}$

E27-1.7 Compressive Deflection

LRFD [14.7.6.3.3, 14.7.5.3.6]

Average vertical compressive stress:

Average compressive stress due to total load

$\boxed{\sigma_S = 0.636}$ ksi

Average compressive stress due to live load

$$\sigma_L := \frac{LL_{serv}}{L \cdot W}$$

$\boxed{\sigma_L = 0.172}$ ksi

Average compressive stress due to dead load

$$\sigma_D := \frac{DL_{serv}}{L \cdot W}$$

$\boxed{\sigma_D = 0.464}$ ksi

Use **LRFD [Figure C14.7.6.3.3-1]** to estimate the compressive strain in the interior and cover layers. Average the values from the 50 Durometer and 60 Durometer curves to obtain values for 55 Durometer bearings.

LAYER	LOAD	S	STRESS (ksi)	50 DUROMETER STRAIN	60 DUROMETER STRAIN	AVERAGE STRAIN
INTERNAL	DEAD LOAD	9.231	0.464	2.3%	2.1%	2.2%
	TOTAL LOAD	9.231	0.636	3.1%	2.7%	2.9%
COVER	DEAD LOAD	18.462	0.464	1.8%	1.5%	1.7%
	TOTAL LOAD	18.462	0.636	2.2%	1.9%	2.1%

Initial compressive deflection of n-internal layers and 2 cover layers under total load:

$\boxed{\varepsilon_{int} = 0.029}$ Compressive strain in the interior layer

$\boxed{\varepsilon_{cover} = 0.021}$ Compressive strain in the cover layer

$\boxed{n = 7}$ layers

$\boxed{h_{ri} = 0.5}$ in

$\boxed{h_{rcover} = 0.25}$ in

$\delta := n \cdot h_{ri} \cdot \varepsilon_{int} + 2 \cdot h_{rcover} \cdot \varepsilon_{cover}$ Modification of **LRFD [Equation 14.7.5.3.6-1]**

$\boxed{\delta = 0.112}$ in

Initial compressive deflection under dead load:

$\boxed{\varepsilon_{intDL} = 0.022}$

$\boxed{\varepsilon_{coverDL} = 0.017}$

$\delta_{DL} := n \cdot h_{ri} \cdot \varepsilon_{intDL} + 2 \cdot h_{rcover} \cdot \varepsilon_{coverDL}$ $\boxed{\delta_{DL} = 0.086}$ in

Deflection due to creep:

$C_d := 0.30$ Average value between 50 and 60 Durometer **LRFD [Table 14.7.6.2-1]**

$\delta_{CR} := C_d \cdot \delta_{DL}$ $\boxed{\delta_{CR} = 0.026}$ in

Compressive deflection due to live load:

$\delta_{LL} := \delta - \delta_{DL}$ $\boxed{\delta_{LL} = 0.027}$ in

Deflection due to creep and live load: **LRFD [C14.7.5.3.6]**

$\delta_{CRLL} := \delta_{CR} + \delta_{LL}$ $\boxed{\delta_{CRLL} = 0.052}$ in

$\boxed{\delta_{CRLL} = \text{"< 0.125 in., OK"}}$

Initial compressive deflection of a single internal layer:

$\varepsilon_{int} \cdot h_{ri} < 0.09 \cdot h_{ri}$ **LRFD [14.7.6.3.3]** $\boxed{\varepsilon_{int} = 0.029}$

$\boxed{\varepsilon_{int} = \text{"< 0.09, OK"}}$

E27-1.8 Anchorage

LRFD [14.8.3]

Shear force generated in the bearing due to temperature movement:

$$H_u := G \cdot A \cdot \frac{\Delta_u}{h_{rt}}$$ **LRFD [Equation 14.6.3.1-2]**

$G := 0.165$ conservative assumption, maximum value of G, ksi

Factored shear deformation of the elastomer

$\Delta_u := \gamma_{TU} \cdot \Delta_s$ $\boxed{\Delta_u = 1.996}$ in

Plan area of elastomeric element

$\boxed{L = 15}$ in $\boxed{W = 24}$ in

$A := L \cdot W$ $\boxed{A = 360}$ in^2

$$H_u := G \cdot A \cdot \frac{\Delta_u}{h_{rt}}$$ $\boxed{H_u = 29.638}$ kips

(This value of H_u can be used for substructure design)

Minimum vertical force due to permanent loads:

$\gamma_{DLserv} := 1.0$

$P_{sd} := \gamma_{DLserv} \cdot (DL_{serv} - DL_{ws})$ $\boxed{P_{sd} = 144}$ kips

$$\sigma := \frac{P_{sd}}{A}$$ $\boxed{\sigma = 0.400}$ ksi

$\boxed{\sigma = \text{"> 0.200 ksi, OK, anchorage is not required per WisDOT exception to AASHTO"}}$

E27-1.9 Reinforcement:

LRFD [14.7.6.3.7, 14.7.5.3.5]

Service limit state:

$h_{max} := h_{ri}$ $\boxed{h_{max} = 0.5}$ in

$\boxed{\sigma_s = 0.636}$ ksi

$\boxed{F_y = 36}$ ksi

$$h_s \geq \frac{3 \cdot h_{max} \cdot \sigma_s}{F_y}$$ **LRFD [Eq 14.7.5.3.5-1]** $\boxed{h_s = 0.125}$ in

$\boxed{\frac{3 \cdot h_{max} \cdot \sigma_s}{F_y} = 0.027}$ in

$\boxed{check = \text{"< hs,   OK"}}$

Fatigue limit state:

$$h_s \geq \frac{2 \cdot h_{max} \cdot \sigma_L}{\Delta F_{TH}}$$ **LRFD [Eq 14.7.5.3.5-2]** $\boxed{\sigma_L = 0.172}$ ksi

$\boxed{h_s = 0.125}$ in

$\Delta F_{TH} := 24.0$ ksi Constant amplitude fatigue threshold for Category A **LRFD [Table 6.6.1.2.5-3]**

$\boxed{\frac{2 \cdot h_{max} \cdot \sigma_L}{\Delta F_{TH}} = 0.007}$ in

$\boxed{check = \text{"< hs,   OK"}}$

E27-1.10 Rotation:

LRFD [14.7.6.3.5, C14.7.6.1]

Design for rotation in Method A is implicit in the geometric and stress limit requirements spelled out for this design method. Therefore no additional rotation calculations are required.

Check requirement for tapered plate: **LRFD [14.8.2]**

Find the angle between the alignment of the underside of the girder and a horizontal line. Consider the slope of the girder, camber of the girder, and rotation due to unfactored dead load deflection.

Inclination due to grade line:

$L_{span} := 150$ Span length, ft

@ pier:

$EL_{Pseat} := 856.63$ Beam seat elevation at the pier, in feet

$h_{Pbrg} := 0.5$ Bearing height at the pier, in

Bottom of girder elevation at the pier, in feet

$$EL_1 := EL_{Pseat} + \frac{h_{Pbrg}}{12} \qquad \boxed{EL_1 = 856.672}$$

@ abutment:

$EL_{Aseat} := 853.63$ Beam seat elevation at the abutment, in feet

$t_{plate} := 1.5$ Steel top plate thickness, in

$\boxed{H = 5}$ Total elastomeric bearing height, in

Total bearing height, at the abutment, in

$h_{Abrg} := H + t_{plate}$ $\boxed{h_{Abrg} = 6.5}$ in

Bottom of girder elevation in feet

$$EL_2 := EL_{Aseat} + \frac{h_{Abrg}}{12} \qquad \boxed{EL_2 = 854.172}$$

Slope of girder

$$S_{GL} := \frac{\left|EL_1 - EL_2\right|}{L_{span}} \qquad \boxed{S_{GL} = 0.017}$$

Inclination due to grade line in radians

$\theta_{GL} := \operatorname{atan}(S_{GL})$ $\boxed{\theta_{GL} = 0.017}$ radians

Inclination due to residual camber:

$\Delta_{camber} := 3.83$ Maximum camber of girder, in

$\Delta_{DL} := 2.54$ Maximum dead load deflection, in

$\Delta_{LL} := 0.663$ Maximum live load deflection, in

Residual camber, in

$\Delta_{RC} := \Delta_{camber} - \Delta_{DL}$ $\boxed{\Delta_{RC} = 1.290}$ in

To determine the slope due to residual camber, use a straight line from C/L Bearing to the 1/10 point. Assume that camber at 1/10 point is 40% of maximum camber (at midspan).

$S_{RC} := \dfrac{0.4 \cdot \Delta_{RC}}{0.1 \cdot L_{span} \cdot 12}$ Slope due to residual camber $\boxed{S_{RC} = 0.003}$

Inclination due to residual camber in radians

$\theta_{RC} := \operatorname{atan}(S_{RC})$ $\boxed{\theta_{RC} = 0.003}$ radians

Total inclination due to grade line and residual camber in radians

$\theta_{SX} := \theta_{GL} + \theta_{RC}$ $\boxed{\theta_{SX} = 0.020}$ radians

$\boxed{\theta_{SX} = \text{"> 0.01 rad, NG, top plate must be tapered"}}$

(The plate should also be tapered if θ_{SX} x L_P >= 1/8")

Top plate dimensions:

$\boxed{t_{plate} = 1.5}$ Minimum thickness of top plate, in

$L_p := L + 2$ Length of top plate

$\boxed{L_p = 17}$ in

$\boxed{L_p \cdot \theta_{SX} = 0.332}$ in

Thickness of top plate on thicker edge

$t_{pmax} := t_{plate} + L_p \cdot \tan(\theta_{SX})$ $\boxed{t_{pmax} = 1.832}$ in

E27-1.11 Bearing summary:

Laminated Elastomeric Bearing Pad:

Length = 15 inches

Width = 24 inches

Steel reinforcing plates: 8 @ 1/8"

Internal elastomer layers: 7 @ 1/2"

Cover elastomer layers: 2 @ 1/4"

Total pad height: 5"

<u>Steel Top Plate (See standard detail):</u>

Length = 17 inches

Width = 30 inches

Thickness = 1 1/2" to 1 7/8"

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28.1 Introduction

Many structures have joints that must be properly designed and installed to insure their integrity and serviceability. Bridges as well as highway pavements, airstrips, buildings, etc. need joints to take care of expansion and contraction caused by temperature changes. However, bridges expand and contract more than pavement slabs or buildings and have their own special types of expansion devices.

Current practice is to limit the number of bridge expansion joints. This practice results in more movement at each joint. There are so many potential problems associated with joints that fewer joints are recommended practice. Expansion joints are placed on the high end of a bridge if only one joint is placed on the bridge. This is done to prevent the bridge from creeping downhill and to minimize the amount of water passing over the joint.

Open joints generally lead to future maintenance. Water and debris fall through the joint. Water running through an open joint erodes the soil under the structure, stains the bent cap and columns, and leads to corrosion of adjacent girders, diaphragms, and bearings. During freeze-thaw conditions, large icicles may form under the structure or ice may form on the roadway presenting a traffic hazard. Debris acts with water in staining the substructure units and plugs the drainage systems.

In the past, open steel finger type joints were used on long span bridges where large movements encountered. Finger joints were placed in the span near the point of contraflexure and were placed on the structure where they are required structurally. Drains were located to prevent drainage across the joint if feasible. In some areas, they were provided with a drainage trough to collect the water passing through.

Sliding steel plate joints are semi-open joints since water and light debris can pass through. A sealant placed in the joint prevents some water from passing through. It also prevents the accumulation of debris which can keep the joint from moving as it was designed. To date, considerable maintenance has occurred with sealants and neoprene troughs have been added to collect the water at some sites.

Currently finger and sliding plate details are maintained for joint maintenance and retrofitting but are not used for new structures. Watertight expansion devices such as strip seals and modular types are recommended for new structures. Although these expansion joints are not completely watertight; they have been effective in reducing damage to adjacent girders, diaphragms, bearings and substructure units.

The neoprene compression seal is a closed joint which is watertight if it is properly installed and an adequate adhesive is employed. Compression seals are only used for fixed joints. Strip-seals are watertight joints which are used in place of sliding plate joints or finger joints in an attempt to keep water and debris on the bridge deck surface.

Refer to Figure 12.7-1 for placement of expansion devices. The following criteria is used for placement of expansion devices:

28.1.1 General

Use watertight expansion joints wherever possible according to the design criteria and of all structure lengths. On skews over 45°, strip seals must be oversized to compensate for racking of the joint. For thermal movements greater than 4 inches modular expansion devices are recommended.

28.1.2 Concrete Spans

An expansion device is required if the expansion length of the structure exceeds 300 feet. At this point the geometrics of the structure determine the number of expansion joints required with a maximum expansion length of 400 feet.

As an example, consider a prestressed girder structure 700 feet long on flexible piers and 0° skew. Considering the two piers near the center of the span as fixed, the structure can expand toward each abutment with maximum expansion lengths less than 400 feet. A 400 series model strip seal expansion joint at each abutment is adequate for this structure.

28.1.3 Steel Spans

Watertight joints are required on all painted and unpainted steel structures to control staining of the substructure units due to corrosion of the steel girders, diaphragms, and bearings.

See Figure 12.7-1 to determine the appropriate abutment type and, hence, whether expansion devices are required. The geometry of the structure determines the number of expansion devices required and the amount of movement at each device. Some factors to consider are temperature expansion with high skew angles may cause "racking" of the structure; higher abutments have more uncertainty to movement due to backfill pressure; and curved girders add torsional and shear forces.

Long span structures on tall flexible piers may have much longer expansion lengths than short span structures on short rigid piers. The longer spans have much less resistance to horizontal temperature movement caused by bearing friction and pier rigidity. These types of structures are designed for joint movements of 4 inches or greater using modular expansion devices.

28.1.4 Thermal Movement

The maximum thermal movement required at expansion joints is based on the following table:

Structure Type	Temperature Range	Thermal Coefficient
Steel:	-30 to 120°F	0.0000065/F
Concrete:	+5 to 85°F	0.0000060/F
*Prestressed Girder:	+5 to 85°F	0.0000060/F

Table 28.1-1
Thermal Movement

* For Prestressed girders add shrinkage due to creep of .0003 ft/ft. This value should be used in setting the joint opening as the joint opening will continue to widen over time.

The expansion length is measured along the centerline of the bridge and the length is normal to the joint opening for structures with a zero skew. The designer should provide adequately sized joints (i.e. round up in size if between two joint sizes or use additional joints or a different type of joint).

The annual mean temperature for Wisconsin is 45 °F. For the setting of strip seal expansion devices, see Standard for Strip Seal Expansion Joint Details for the joint opening when the expansion length is less than or equal to 230 feet. When the expansion length is greater than 230 feet show a temperature table with the joint openings from 5°F to 85°F in 10°F increments.

Note that the neutral point for temperature movement is not always located at the fixed pier. See Chapter 13 – Piers for an explanation of how to calculate the neutral point.

28.2 Compression Seals

28.2.1 Description

This is a preformed, compartmented, elastomeric polychloropene (neoprene) device. In the past, compression seals were used sparingly on fixed joints provided there was little or no movement of the joint. However, compression seals shall no longer be used in this application due to the fact that the seals tend to leak over time. Compression seals shall be used only in longitudinal construction joint locations or for rehabilitation projects that do not involve full joint replacement (i.e., where the existing seal has pulled out of the joint and needs to be replaced).

28.2.2 Joint Design

Most applications have been for bridge rehabilitation where the seal is installed into the concrete deck without armor.

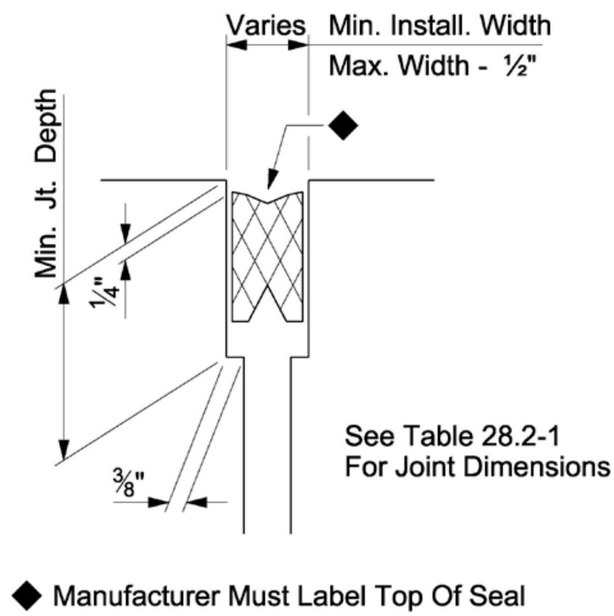


Figure 28.2-1 .
Joint Design

Manufacturer must label top of seal.

28.2.3 Seal Size

The width of the compression seal to be used in a given joint opening is computed by adding the as-constructed joint width plus a small width safety factor. For best results oversize the seal by a minimum of ½ inch. See Table 28.2-1 for approximate dimensions.

28.2.4 Installation

Ease of installation is achieved using a lubricant-adhesive which as the name implies acts initially as a lubricant then cures out to form an adhesive membrane between the contact faces of the angle and seal. This membrane, together with the compressive action of the seal, is designed to provide a waterproof joint interface.

The following information is a guide for the installation of neoprene compressive seals:

1. Thorough cleaning of joint faces is essential. Forced air or manual dusting handles most cases; use a solvent on oily areas.

2. Require application of the manufacturer's lubricant-adhesive to the sides of the neoprene seal as well as the joint faces. An adequate coating of the lubricant-adhesive is helpful in installation.

3. Proper installation tools consist of hand or machine tools that compress and eject the seal or weighted rollers that squeeze it in place. Screwdrivers, pry bars or other sharp ended tools which may puncture the seal are not allowed.

4. Stretching in excess of 5% is not permitted.

5. It is imperative that the seal be installed below the pavement surface. The minimum depth recess to top of seal is ¼ inch.

6. Prior to shipping, all compression seals are to be labeled TOP SIDE by the manufacturers. Field installation reports indicate difficulty in determining TOP SIDE for some types of seals. Also, the seal cross-section is not shown on a shop drawing unless the joint is armored.

28.2.5 Maintenance

Manual removal of incompressible materials which tend to collect within the joint opening is desirable. However, in most cases this is not necessary since the tire forces the material against the elastic neoprene seal which rebounds causing the material to bounce up and out of the seal.

It is essential to the operation of the seal that no form of hot or cold joint filler be placed over the top of the seal. This includes resurfacing mats or overlays. The reasons are as follows:

1. Hot fillers may either melt the seal or seriously affect the elastomeric properties for future performance.

2. The filler acts as a constant media of transmitting undue vertical tire forces to the compression seal which may break the interface bond.

SEAL WIDTH	SEAL HEIGHT	MIN. JOINT WIDTH	MAX. JOINT WIDTH	*MIN. INSTALL. WIDTH	MIN. JOINT DEPTH
2	2 ±	1 ±	1 ¾ ±	1 ¼ ±	2 ⅞ ±
2 ¼	2 ½ ±	1 ±	2 ±	1 ⅜ ±	3 ⅛±
2 ½	2 ¾ ±	1 ¼ ±	2 ¼ ±	1 ⅝ ±	3 ⅝ ±
3	3 ⅜ ±	1 ⅜ ±	2 ½ ±	1 ¾ ±	4 ¼ ±
3 ½	3 ½ ±	1 ½ ±	3 ±	2 ⅛ ±	4 ¾ ±
4	4 ½ ±	1 ¾ ±	3 ½ ±	2 ⅝ ±	5 ½ ±

Table 28.2-1
Approximate Compression Seal Dimensions

* This is the minimum practical limit as suggested by the seal manufacturer.

28.3 Strip Seal Expansion Devices

28.3.1 Description

Strip seal expansion devices are molded neoprene glands inserted and mechanically locked between armored interfaces of extruded steel sections. The name "Strip Seal" is derived from the strip profile of the neoprene seal. During structure movements a preformed central hinge enables the strip seal profile to fold between the seal extrusions. Strip seal design details are given on the Standards for Strip Seal Expansion Joint Details and Strip Seal Cover Plate Details.

Ease of installation is attained by applying a lubricant-adhesive to the steel extrusions; which as the name implies acts initially as a lubricant; then cures to form an adhesive membrane between contact surfaces of the extrusions and neoprene gland. The neoprene glands are generally inserted in the field by using a tire-iron type tool. A minimum transverse roadway surface opening between the extrusions of 1 ½ inches or greater will facilitate the field installation of the neoprene gland. When extra size or travel capacity is available, joint openings can be increased to facilitate gland installation keeping the maximum transverse roadway joint opening at 4 inches for new construction. Greater openings may be used on maintenance projects only.

The strip seal is readily adaptable to changes in interfacial elevations as well as longitudinal skew deformations. The neoprene gland is installed as one continuous length on any given joint application. Additional considerations are given to the "racking" movement on the neoprene gland as the structure skew angle increases.

28.3.2 Curb and Parapet Sections

The strip seal is curved up into the curb or safety parapet with cover plates. The details are shown on Standard for Strip Seal Cover Plate Details. The resulting recess between the parapet and joint requires cover plates for maintenance considerations.

28.3.3 Median and Sidewalk Sections

Median cover plates are not required if the joint is placed at the median surface, otherwise they are required. All sidewalk joints must have cover plates as shown on the standard details.

28.3.4 Size Selection

The first consideration in strip seal size selection is the effective expansion length for the given joint location. Table 28.1-1 is established in accordance with AASHTO Specifications by employing a cold climate temperature range given in 28.1.4 for determining the maximum span lengths for the joint movement limits. The span length was decreased for prestressed girder structures to further accommodate movements due to concrete creep and shrinkage. The “Maximum Expansion Length” for a given joint size and structure type is shown in Table 28.3-1

On new structures and deck replacements, provide details for strip seal models having a minimum size of 4 inches. If the skew angle exceeds 30 degrees, limit the actual racking displacement to 60 percent of the seal's rated capacity or select the next larger size neoprene gland model to reduce stresses caused by racking. For skew angles greater than 45 degrees, limit the actual racking movement to 50 percent of the seal's rated capacity. Some manufacturers provide a 5 inch gland which makes an excellent alternate on installations sized for 4 inches of movement on a high skewed structure. The maximum allowable opening perpendicular to the center line of the joint is 4 inches on all structures.

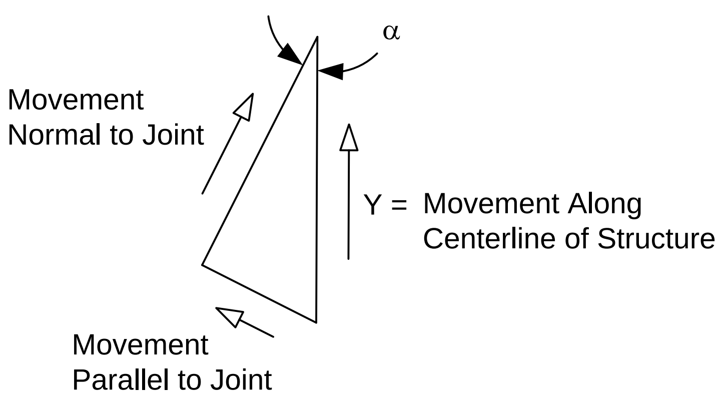
After selecting the proper strip seal model, refer to Table 28.3-1 for the joint opening at the mean shaded underside of deck temperature of 45°F. The dimensions are given normal to the joint opening in the roadway measured between the inside edges of the extrusion on the top sides. Refer to the Strip Seal design example for additional considerations regarding skew angle and joint installation. A minimum transverse roadway joint opening of 1 ½ inches or greater is recommended measured from between the top inside extrusion edges to facilitate the neoprene gland installation and/or replacement.

Performance evaluations of strip seal joints in-service indicate that the neoprene glands are not always installed properly. In some cases, both "ears" of the neoprene lug have not been inserted into the steel extrusion. In other cases, the gland has been installed upside down. As a result, manufacturers are requested to label "Topside" on the neoprene glands prior to shipping.

28.3.4.1.1 Example

Strip Seal Application, minimum size is 4 inch size. Given: Prestressed concrete girder structure having 350 feet of expansion length with a 33 degree skew angle.

From Table 28.3-1, under Prest., select the minimum size 4 inch size and check the racking displacement in accordance with 28.3.4.



<u>Figure 28.3-1</u>
Racking Displacement

$$Y = (350')(12''/')*(6x10^{-6})(80) + (350')(12''/')(0.0003) = 2'' + 1.26'' = 3.26''$$

$$Y(normal) = Y(\cos\alpha) = 3.26 \cos 33^{o} = 2.73''$$

Y(parallel) = Y(sin α) = 3.26 sin 33^{o} = 1.78”

In this case parallel racking as a percentage of joint capacity is 44.5% (< 60%) of the 4 inch model capacity.

Refer to Table 28.3-1 for joint opening at 45°F which is 2 1/4 inches. The opening size should be reduced by 1 ¼ inches to account for future creep and shrinkage, but maintain 1 ½ inch minimum opening. Show the Strip Seal Size on the plans. Approved Joint Manufacturers are shown in the STSP’s.

<table>
<tr><th rowspan="2">Size</th><th rowspan="2">Inch Travel</th><th colspan="3">Max. Expansion Lengths (feet)</th><th rowspan="2">Jt. Opening @ 45°F (inch)</th></tr>
<tr><th>Conc</th><th>Prest</th><th>Steel</th></tr>
<tr><td>4-Inch</td><td>± 2</td><td>615</td><td>380</td><td>300</td><td>2 ¼</td></tr>
<tr><td>5-Inch</td><td colspan="5">Recommended for expansion movement requirements ranging from 3” to 4” on skews greater than 30 degrees. Use the same criteria as with the 4-inch models.</td></tr>
<tr><td colspan="6">The joint opening at 45°F is given at mean shaded underside deck temperature normal to the joint for zero degree skew of structure. Show joint openings from 5°F to 85°F in 10°F increments if the expansion length exceeds 230 feet. For prestressed girders the joint opening should be reduced by the amount calculated for future creep and shrinkage. A minimum opening of 1 ½ inches is required for setting.</td></tr>
</table>

Table 28.3-1
Expansion Joint Openings

28.4 Steel Expansion Joints

With the availability of modular watertight joints having 3 inch increments of expansion capacity and greater, steel expansion devices are becoming less attractive. Positive protection against expansion joint leakage is required to prevent deterioration of bridge bearings and supporting substructure units. Steel expansion joints can be made watertight by using neoprene troughs. Past experience indicates that maintenance is required on a routine basis to keep the drain troughs free of debris. However, steel expansion devices with neoprene troughs are occasionally detailed on designated projects.

28.4.1 Plate Type Expansion Joint

The plate type expansion joint is limited to structures having relatively small thermal movements. The plate type expansion joint is generally limited to movements less than 2 ½ inches. When this joint is inspected before installation, examine the joint for warpage with the plates lying together loose and not bolted. When the plates are bolted, it is difficult to detect plate warpage. There are maintenance problems such as deterioration of the joint fillers and sliding plates resulting in joint leakage.

28.4.2 Finger Type Expansion Joint

The finger type expansion joint is recommended for structures requiring thermal movements greater than 4 inches. The plate girder finger joint details are shown in Chapter 40.

Expansion joint supports are detailed under the roadway portion of the deck at each girder. When the exterior girder is positioned under a curb section, a support is detailed off the end diaphragm approximately 20 inches from the face of the curb. If the girder spacing or magnitude of the skew angle creates a length of expansion joint greater than 12 feet between adjacent girders; an intermediate support is placed off each end diaphragm at its midspan.

An optional field welded transverse joint is permitted on all steel expansion joints which are detailed over 34 feet in length. The joint location or weld details are not shown on the bridge plans; actual fabrication details are approved on the shop drawings.

Prior to the deck pour, a minimum blockout of 5 feet on each side of the joint is required for finger type expansion joints. This procedure eliminates rotation of the pre-set expansion joint during the deck pour. The finger joint is set and the blocked-out section is poured after the deck pour.

28.5 Modular Expansion Devices

28.5.1 Description

Modular expansion devices consist of molded elastomeric seals which are mechanically locked between steel separation beams. The name "Modular" is used due to the configuration which incorporates a series of standard units. Each unit can accommodate 3 inches of movement; up to 30 inches of movement normal to the joint can be provided. The separation beams are supported by individual support beams; welding provides a permanent contact. The support beam is held down by its extremities at the bearings and is seated within the support box. The support boxes are to be constructed with a minimum steel plate thickness of ½ inch.

The steel separation beams are spaced uniformly via a system of springs that counter the forces exerted on the seals. The springs are arranged such that they will be compressed when the joint is open and the seals are extended. They will relax as the elastomeric seals go into compression due to a rising temperature. Separation beams shall be designed for vertical load of AASHTO HS20 Live Loading plus a minimum of 30 percent for impact and a horizontal load of 50% of vertical load. Specifications should include fatigue testing of weld details for separation beam to support beam connections.

The joint should be designed for 100,000,000 fatigue cycles. All joints should be tested and certified that they meet the loading requirements. Modular expansion devices are prefabricated as a single unit and transported to the site. Generally the anticipated joint opening is preset during fabrication and held in place with threaded rods. If the field temperature varies by more than 10°F from the preset temperature; the joint opening is reset just prior to the closure pour. Refer to Figure 28.5-1 showing the strip-seal type neoprene gland element. The elastomeric neoprene box or strip seals are installed as one continuous length on any given joint application. In all cases, the modular expansion device is carried through the curb line without any change in direction and turned up at their extremities. Cover plates are detailed to cover and transition the gap on sidewalks rand other areas as needed.

Manufacturers recommend sizing the modular expansion device for the calculated movement perpendicular to the joint opening. Also, this recommendation is made for skewed structures. However, consideration should be given to selecting the next higher 3 inch capacity joint where skews are involved. This cost is nominal in comparison to the benefits gained from reducing the racking movement and stress in the seal parallel to the joint opening.

Research indicates that continuous modular expansion devices eliminate possible points of leakage by not having surfaces that have to be sealed. The higher installation costs of modular systems are offset by their greater capacity, improve performance, and reduced future maintenance costs.

Some construction details are recommended for long term performance of modular expansion devices. Minimum thickness of the separation beams, anchor beams and plates holding the equalizers is ¾ inch. Full penetration welds should be used between the separation beams and individual support beams. All joined surfaces should be welded, this applies mainly to the support boxes. Use maximum spacing of 8 feet to support the device during deck construction.

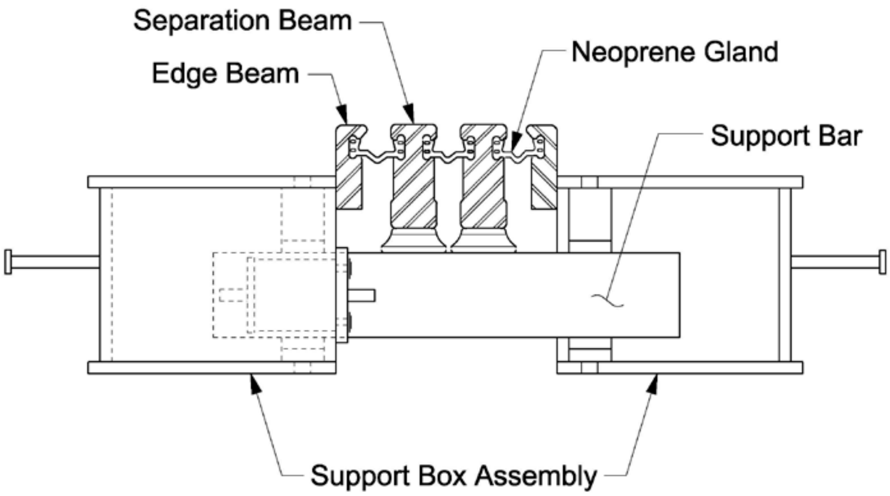


Figure 28.5-1
Modular Expansion Device

28.5.2 Size Selection

The first consideration is the effective span length for computing total thermal movement at the given joint location. Table 28.5-1 is established in accordance with AASHTO Specifications for a cold climate temperature range of -30° to 120°F. for steel girder structures. For preliminary design, maximum span lengths in Table 28.5-1 may be increased by 25 percent for multi-span prestressed girder structures. A more exact analysis shall be made for prestressed girder structures taking into account the shortening due to creep and shrinkage of the concrete. The maximum expansion length, block out depths, and width requirements for a given joint size vary by manufacturer as the transverse separation beams vary in top flange width. Final construction details are to be as shown on approved shop drawings.

As an example, the size selections for a steel girder structure having an expansion length of 720 feet and a zero degree skew are the 3 cell models. However, the next size joint should be considered as it is desirable to allow 1 inch and preferably 2 inches extra movement for construction discrepancies. The strip-seal as an alternate sealing element has the advantage of being easier to install, allows a lower height of joint, and offers excellent tear resistance when reinforced.

After selecting the proper modular expansion device size, refer to Table 28.5-1 for the required clear opening between all flange tips at the mean temperature of 45°F. (Z = 1+2+3) Manufacturers of modular expansion joints recommend setting the joint opening just prior to completing the concrete pour.

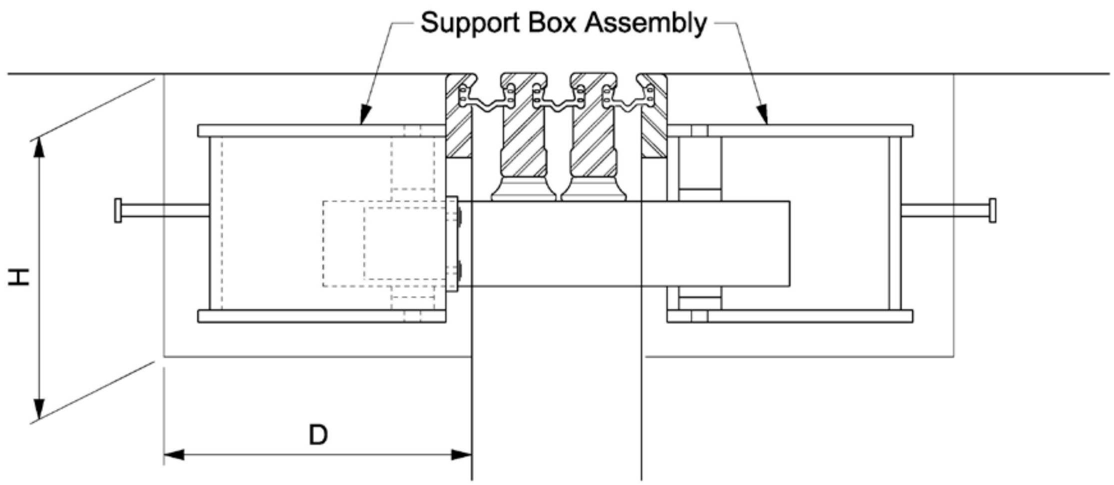


Figure 28.5-2
Modular Expansion Device Dimensions

Number of Cells	Max.Thermal Movement (inch)	Maximum Expansion Lengths (1) (feet)	Expansion Device Settings @ 45°F (2) (inch) Z	Standard Dimensions (2) (inch)	
				H	D
2	6	520	3	17 ±	18 ±
3	9	780	4 ½	17 ±	21 ±
4	12	1040	6	18 ±	25 ±

Table 28.5-1
Joint Clear Opening

1. Maximum expansion range is based on steel girder structures in cold climate temperatures; -30 to 120°F.

2. The joint opening shown as Z for 45°F is taken at mean shaded underside of deck temperature normal to the joint for zero degree skew of structure. Separation beam flange widths vary between manufacturers and these values are given for total opening, actual dimensions shall be verified from manufacturer's standard details or shop drawings. See Figure 28.5-2

Coping of wide flanged prestressed concrete girders may be necessary to facilitate placement of the support boxes.

28.6 Joint Performance

Currently the approved modular expansion devices with continuous neoprene seals and individual bearing support bars have performed well. From the maintenance standpoint, they are preferred over steel finger joints with troughs that require periodic cleaning. Galvanizing modular expansion joints is required due to the number of steel components subjected to chlorides and potential for corrosion. Strip seal joints require galvanizing too.

Joint cleaning and inspection/repair of the neoprene glands is imperative to insure long-term joint performance.

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29.1 General

Wherever practical, bridge drainage should be carried off the structure along the curb or gutter line and collected with roadway catch basins. Floor drains are not recommended for structures less than 400' long and floor drain spacing is not to exceed 500' on any structure. However, additional floor drains are required on some structures due to flat grades, superelevations and the crest of vertical curves. The drains are spaced according to the criteria as set forth in 29.2, which includes acceptable spread of water measured from gutterline as a function of design speed, design storm frequency and duration of rainfall. Additional drains should not be provided other than what is required by design. Utilizing blockouts in parapets to facilitate drainage is not allowed.

Superelevation on structures often creates drainage problems other than at the low point especially if a reverse curve is involved. Water collects and flows down one gutter and as it starts into the superelevation transition it spreads out over the complete width of roadway at the point of zero cross-slope. From this point the water starts to flow into the opposite gutter. Certain freezing conditions can cause traffic accidents to occur in the flat area between the two transitions. To minimize the problem, locate the floor drain as close to the cross over point as practical. Floor drains are installed as near all joints as practical to prevent gutter flow from passing over and/or through the joints.

The Bureau of Structures recommends the Type “GC” floor drain for new structures. Type "GC" floor drains are gray iron castings that have been tested for hydraulic efficiency. Where hydraulic efficiency or girder flange to edge of deck geometry dictates the use of a different floor drain configuration, BOS recommends the Type “WF” floor drain. Steel fabricated floor drains Type “H” provide an additional 6” of downspout clearance and are retained for maintenance of structures where floor drain size modifications are necessary.

All of the floor drains shown on the Standards have grate inlets. When the longitudinal grade exceeds 1 percent, hydraulic flow testing indicates grates with rectangular longitudinal bars are more efficient than grates having transverse rectangular bars normal to flow. However, grates with bars parallel to the direction of traffic are hazardous to bicyclists and even motorcyclists as bar spacing is increased for hydraulic efficiency. As a result, transverse bars sloped toward the direction of flow are detailed for the cast iron floor drains.

Downspouts are to be fabricated from reinforced thermosetting resin (fiberglass) pipe having a diameter not less than 6” for all new structures. Galvanized standard pipe or reinforced fiberglass material may be used for downspouts when adjusting or rehabilitating existing floor drains. Downspouts are required on all floor drains to prevent water and/or chlorides from getting on the girders, bearings, substructure units, etc. Downspouts should be detailed to extend a minimum of 6” below low prestressed girder bottom flange or 1' below low steel to prevent flange or web corrosion. A downspout collector system is required on all structures over grade separations. Reinforced fiberglass pipe is recommended for all collector systems due to its durability and economy. In the design of collector systems, elimination of unnecessary bends and provision for an adequate number of clean outs is recommended.

29.2 Design Criteria

The flow of water in an open channel depends on its cross section, grade, and roughness. Generally, the gutter cross section on a structure is right triangular in shape with the curb, median or parapet forming the vertical leg. For design speeds 45 mph or less, floor drains are spaced at a distance such that the maximum gutter flow is restricted to a spread width of the shoulder plus one-half the adjacent through driving lane for a given design frequency storm. This defines the hypotenuse of the triangle if the shoulder and driving lane slope are equal. For design speeds greater than 45 mph, floor drains are spaced at a distance such that the maximum gutter flow is restricted to a spread width of the shoulder. An increase in longitudinal and transverse slope increases inlet capacity. In design, it is assumed that all of the water passing over the width of the inlet is taken by that inlet, the remaining water (Q bypass) continues to the next inlet.

For design a storm frequency of 10 years with a duration of 5 minutes is used. This gives an average rainfall intensity (i) of approximately 6" per hour in Wisconsin. A run-off coefficient (C) of 0.9 is used for concrete surfaces.

The Rational Method (English Units) converts rainfall intensity for a given design frequency storm to run-off by the following equation:

$$Q = C\,i\,A$$

Where:

Q = peak rate of run-off in cfs.

C = run-off coefficient for surface type.

i = rainfall intensity in inches/hour.

A = drainage area in acres = $\frac{LW}{43560}$

Where:

L = floor drain spacing in feet.

W = contributing structure width in feet.

The Manning equation modified for triangular flow is used to compute Q and Q_{bypass} for the given gutter section. The modified equation is:

$$Q = 0.56\left(\frac{Z}{n}\right)(S_o)^{\frac{1}{2}}(d)^{\frac{8}{3}}$$

Where:

Q	=	discharge in cfs.
Z	=	reciprocal of cross slope.
n	=	Manning's coefficient of roughness, use n = 0.014 for concrete.
S_o	=	longitudinal slope in feet/foot.
d	=	depth of flow at the deepest point (gutter line) in feet.

Refer to Table 29.2-1, Table 29.2-2 and Table 29.2-3 for values of (Z/n) and to Figure 29.2-1 for a nomographic solution to the Manning equation.

CROSS SLOPE, Sc			1/Sc	VALUES OF Z/n				
				n				
in/ft	in/ft	ft/ft	Z	0.012	0.013	0.014	0.015	0.016
	0.0120	0.0010	1000.00	83,333	76,923	71,429	66,667	62,500
1/64	0.0156	0.0013	768.00	64,000	59,077	54,857	51,200	48,000
	0.0240	0.0020	500.00	41,667	38,462	35,714	33,333	31,250
1/32	0.0313	0.0026	384.00	32,000	29,538	27,429	25,600	24,000
	0.0360	0.0030	333.33	27,778	25,641	23,810	22,222	20,833
	0.0480	0.0040	250.00	20,833	19,231	17,857	16,667	15,625
	0.0600	0.0050	200.00	16,667	15,385	14,286	13,333	12,500
1/16	0.0625	0.0052	192.00	16,000	14,769	13,714	12,800	12,000
	0.0720	0.0060	166.67	13,889	12,821	11,905	11,111	10,417
	0.0840	0.0070	142.86	11,905	10,989	10,204	9,524	8,929
3/32	0.0938	0.0078	128.00	10,667	9,846	9,143	8,533	8,000
	0.0960	0.0080	125.00	10,417	9,615	8,929	8,333	7,813
	0.1000	0.0083	120.00	10,000	9,231	8,571	8,000	7,500
	0.1080	0.0090	111.11	9,259	8,547	7,937	7,407	6,944
	0.1200	0.0100	100.00	8,333	7,692	7,143	6,667	6,250
1/8	0.1250	0.0104	96.00	8,000	7,385	6,857	6,400	6,000
	0.1320	0.0110	90.91	7,576	6,993	6,494	6,061	5,682
	0.1440	0.0120	83.33	6,944	6,410	5,952	5,556	5,208
5/32	0.1563	0.0130	76.80	6,400	5,908	5,486	5,120	4,800
	0.1680	0.0140	71.43	5,952	5,495	5,102	4,762	4,464
	0.1800	0.0150	66.67	5,556	5,128	4,762	4,444	4,167
3/16	0.1875	0.0156	64.00	5,333	4,923	4,571	4,267	4,000
	0.1920	0.0160	62.50	5,208	4,808	4,464	4,167	3,906
	0.2000	0.0167	60.00	5,000	4,615	4,286	4,000	3,750
	0.2040	0.0170	58.82	4,902	4,525	4,202	3,922	3,676
	0.2160	0.0180	55.56	4,630	4,274	3,968	3,704	3,472
7/32	0.2188	0.0182	54.86	4,571	4,220	3,918	3,657	3,429
	0.2280	0.0190	52.63	4,386	4,049	3,759	3,509	3,289
	0.2400	0.0200	50.00	4,167	3,846	3,571	3,333	3,125
1/4	0.2500	0.0208	48.00	4,000	3,692	3,429	3,200	3,000
9/32	0.2813	0.0234	42.67	3,556	3,282	3,048	2,844	2,667
19/64	0.2969	0.0247	40.42	3,368	3,109	2,887	2,695	2,526
	0.3000	0.0250	40.00	3,333	3,077	2,857	2,667	2,500

Table 29.2-1
Values of Z/n for Manning's Equation

CROSS SLOPE, Sc			1/Sc	VALUES OF Z/n				
				n				
in/ft	in/ft	ft/ft	Z	0.012	0.013	0.014	0.015	0.016
5/16	0.3125	0.0260	38.40	3,200	2,954	2,743	2,560	2,400
21/64	0.3281	0.0273	36.57	3,048	2,813	2,612	2,438	2,286
11/32	0.3438	0.0286	34.91	2,909	2,685	2,494	2,327	2,182
	0.3600	0.0300	33.33	2,778	2,564	2,381	2,222	2,083
3/8	0.3750	0.0313	32.00	2,667	2,462	2,286	2,133	2,000
	0.4000	0.0333	30.00	2,500	2,308	2,143	2,000	1,875
13/32	0.4063	0.0339	29.54	2,462	2,272	2,110	1,969	1,846
	0.4200	0.0350	28.57	2,381	2,198	2,041	1,905	1,786
7/16	0.4375	0.0365	27.43	2,286	2,110	1,959	1,829	1,714
15/32	0.4688	0.0391	25.60	2,133	1,969	1,829	1,707	1,600
	0.4800	0.0400	25.00	2,083	1,923	1,786	1,667	1,563
1/2	0.5000	0.0417	24.00	2,000	1,846	1,714	1,600	1,500
17/32	0.5313	0.0443	22.59	1,882	1,738	1,613	1,506	1,412
	0.5400	0.0450	22.22	1,852	1,709	1,587	1,481	1,389
9/16	0.5625	0.0469	21.33	1,778	1,641	1,524	1,422	1,333
19/32	0.5938	0.0495	20.21	1,684	1,555	1,444	1,347	1,263
	0.6000	0.0500	20.00	1,667	1,538	1,429	1,333	1,250
5/8	0.6250	0.0521	19.20	1,600	1,477	1,371	1,280	1,200
21/32	0.6563	0.0547	18.29	1,524	1,407	1,306	1,219	1,143
	0.6600	0.0550	18.18	1,515	1,399	1,299	1,212	1,136
11/16	0.6875	0.0573	17.45	1,455	1,343	1,247	1,164	1,091
	0.7000	0.0583	17.14	1,429	1,319	1,224	1,143	1,071
23/32	0.7188	0.0599	16.69	1,391	1,284	1,192	1,113	1,043
	0.7200	0.0600	16.67	1,389	1,282	1,190	1,111	1,042
3/4	0.7500	0.0625	16.00	1,333	1,231	1,143	1,067	1,000
	0.7800	0.0650	15.38	1,282	1,183	1,099	1,026	962
25/32	0.7812	0.0651	15.36	1,280	1,182	1,097	1,024	960
	0.8000	0.0667	15.00	1,250	1,154	1,071	1,000	938
13/16	0.8125	0.0677	14.77	1,231	1,136	1,055	985	923
	0.8400	0.0700	14.29	1,190	1,099	1,020	952	893
27/32	0.8438	0.0703	14.22	1,185	1,094	1,016	948	889
	0.8500	0.0708	14.12	1,176	1,086	1,008	941	882
7/8	0.8750	0.0729	13.71	1,143	1,055	980	914	857

Table 29.2-2
Values of Z/n for Manning's Equation

CROSS SLOPE, Sc			1/Sc	VALUES OF Z/n				
				n				
in/ft	in/ft	ft/ft	Z	0.012	0.013	0.014	0.015	0.016
	0.9000	0.0750	13.33	1,111	1,026	952	889	833
39/32	1.2188	0.1016	9.85	821	757	703	656	615
15/16	0.9375	0.0781	12.80	1,067	985	914	853	800
	0.9500	0.0792	12.63	1,053	972	902	842	789
	0.9600	0.0800	12.50	1,042	962	893	833	781
31/32	0.9688	0.0807	12.39	1,032	953	885	826	774
1	1.000	0.0833	12.00	1,000	923	857	800	750
	1.020	0.0850	11.76	980	905	840	784	735
	1.080	0.0900	11.11	926	855	794	741	694
	1.140	0.0950	10.53	877	810	752	702	658
	1.200	0.1000	10.00	833	769	714	667	625
2	2.000	0.1667	6.000	500	462	429	400	375
	2.400	0.2000	5.000	417	385	357	333	313
3	3.000	0.2500	4.000	333	308	286	267	250
	3.600	0.3000	3.333	278	256	238	222	208
4	4.000	0.3333	3.000	250	231	214	200	188
	4.800	0.4000	2.500	208	192	179	167	156
5	5.000	0.4167	2.400	200	185	171	160	150
6	6.000	0.5000	2.000	167	154	143	133	125
7	7.000	0.5833	1.714	143	132	122	114	107
	7.200	0.6000	1.667	139	128	119	111	104
8	8.000	0.6667	1.500	125	115	107	100	94
	8.400	0.7000	1.429	119	110	102	95	89
9	9.000	0.7500	1.333	111	103	95	89	83
	9.600	0.8000	1.250	104	96	89	83	78
10	10.00	0.8333	1.200	100	92	86	80	75
	10.80	0.9000	1.111	93	85	79	74	69
11	11.00	0.9167	1.091	91	84	78	73	68
	11.50	0.9583	1.043	87	80	75	70	65
12	12.00	1.0000	1.000	83	77	71	67	63

Table 29.2-3
Values of Z/n for Manning's Equation

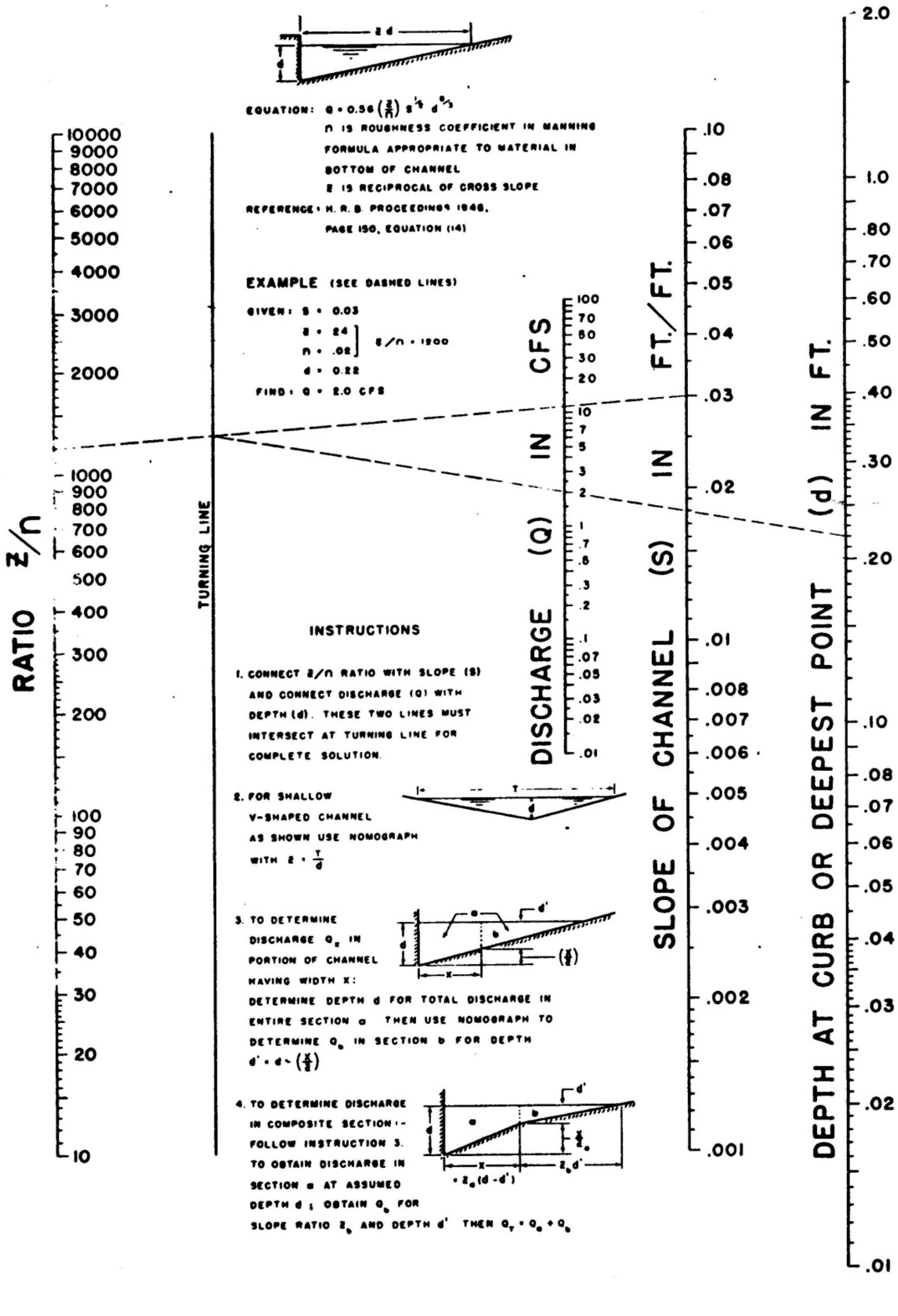

Figure 29.2-1

Nomograph for Flow in Triangular Channels

Modified Manning Solution

29.3 Design Example

The following method is used to compute floor drain spacing by equating net discharge to the Rational Method:

Given: Structure 1200 feet long on a 0.3% grade having a cross slope of 0.02 feet/foot with a contributing structure width of 23'-6". Use Type "GC" floor drain.

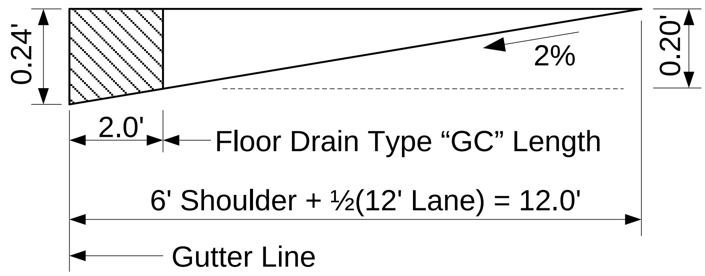

Figure 29.3-1
Cross Section of Flow

Compute: Floor drain spacing

From Table 29.2-1 with a cross slope of 0.02 feet/foot

(Z/n) = 3571.

From Figure 29.2-1, Q = 2.44 cfs and Q_{bypass} = 1.50 cfs.

$$L = (Q - Q_{bypass})\frac{43560}{CiW}$$

$$L = (2.44 - 1.5)\frac{43560}{0.9 \cdot 6 \cdot 23.5}$$

L = 323 ft

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30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates were based on significant changes in the vehicle fleet, the emergence of many new barrier designs, increased interest in matching safety performance to levels of roadway utilization, new policies requiring the use of safety belts, and advances in computer simulation and other evaluation methods.

NCHRP Report 350 differs from NCHRP Report 230 in the following ways: it is presented in all-metric documentation, it provides a wider range of test procedures to permit safety performance evaluations for a wider range of barriers, it uses a pickup truck as the standard test vehicle in place of a passenger car, it defines other supplemental test vehicles, it includes a broader range of tests to provide a uniform basis for establishing warrants for the application of roadside safety hardware that consider the levels of use of the roadway facility, it includes guidelines for selection of the critical impact point for crash tests on redirecting-type safety hardware, it provides information related to enhanced measurement techniques related to occupant risk, and it reflects a critical review of methods and technologies for safety-performance evaluation.

In May of 1997, a memorandum from Dwight A. Horne, the FHWA Chief of the Federal-Aid and Design Division, on the subject of “Crash Testing of Bridge Railings” was published. This memorandum identified 68 crash-tested bridge rails, consolidated earlier listings, and established tentative equivalency ratings that related previous NCHRP Report 230 testing to NCHRP Report 350 test levels.

In 2009, AASHTO published the *Manual for Assessing Safety Hardware* (MASH). MASH is an update to, and supersedes, NCHRP Report 350 for the purposes of evaluating new safety hardware devices. AASHTO and FHWA jointly adopted an implementation plan for MASH that stated that all highway safety hardware accepted prior to the adoption of MASH – using criteria contained in NCHRP Report 350 – may remain in place and may continue to be manufactured and installed. In addition, highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. However, new highway safety hardware not previously evaluated must utilize MASH for testing and evaluation. MASH represents an update to crash testing requirements based primarily on changes in the vehicle fleet.

All bridge railings as detailed in the Wisconsin LRFD Bridge Standard Detail Drawings in Chapter 30, have been approved by FHWA per the crash tests as recommended in NCHRP Report 350. In order to use railings other than Bridge Office Standards, the railings must conform to MASH or must be crash tested rails which are available from the FHWA office. Any railings that are not crash tested must be reviewed by FHWA when they are used on a bridge, culvert, retaining wall, etc.

WisDOT policy states that railings that meet the criteria for Test Level 3 (TL-3) or greater shall be used on NHS roadways and all functional classes of Wisconsin structures (Interstate Highways, United States Highways, State Trunk Highways, County Trunk Highways, and Local Roadways) where the design speed exceeds 45 mph. Railings that meet Test Level 2 (TL-2) criteria may be used on non-NHS roadways where the design speed is 45 mph or less.

There may be unique situations that may require the use of a MASH or NCHRP Report 350 crash-tested railing of a different Test Level; a railing design using an older crash test methodology; or a modified railing system based on computer modeling, component testing, and or expert opinion. These unique situations will require an exception to be granted by the Bureau of Project Development and/or the Bureau of Structures. It is recommended that coordination of these unique situations occur early in the design process.

30.2 Railing Application

The primary purpose of bridge railings shall be to contain and redirect vehicles and/or pedestrians using the structure. In general, there are three types of bridge railings – Traffic Railings, Combination Railings, and Pedestrian Railings. The following guidelines indicate the typical application of each railing type:

1. Traffic Railings shall be used when a bridge is used exclusively for highway traffic.

   Traffic Railings can be composed of, but are not limited to: single slope concrete parapets, sloped face concrete parapets, vertical face concrete parapets, tubular steel railings, and timber railings.

2. Combination Railings can be used concurrently with a raised sidewalk on roadways with a design speed of 45 mph or less.

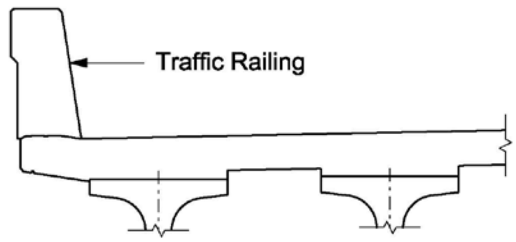
   Combination Railings can be composed of, but are not limited to: single slope concrete parapets with chain link fence, vertical face concrete parapets with tubular steel railings such as type 3T, and aesthetic concrete parapets with combination type C1-C6 railings.

3. Pedestrian Railings can be used at the outside edge of a bridge sidewalk when a Traffic Railing is used concurrently to separate highway and pedestrian traffic.

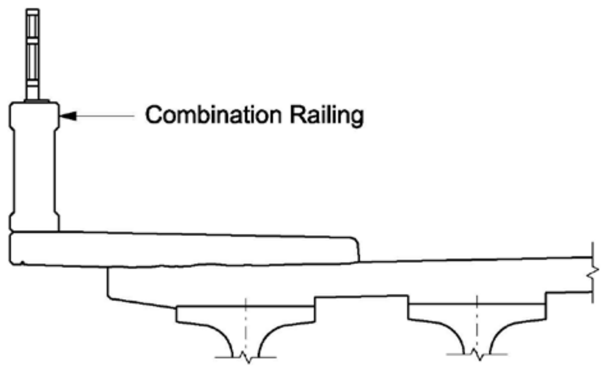
   Pedestrian Railings can be composed of, but are not limited to: chain link fence, tubular screening, vertical face concrete parapets with combination type C1-C6 or type 3T railings, and single slope concrete parapets.

See Figure 30.2-1 below for schematics of the three typical railing types.

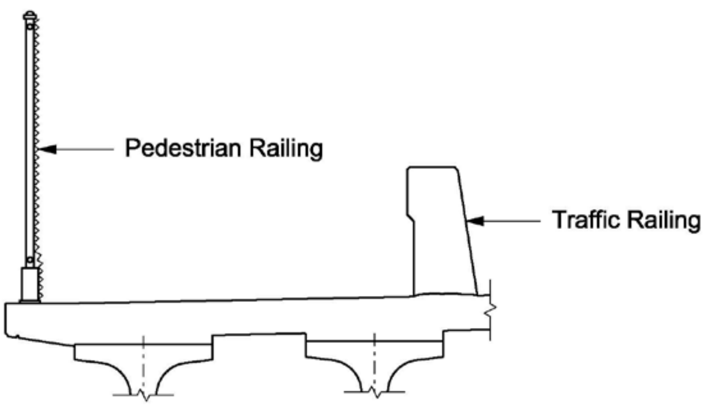
Note that the railing types shown in Figure 30.2-1 shall be employed as minimums. At locations where a Traffic Railing is used at the traffic side of a sidewalk at grade, a Combination Railing may be used at the edge of deck in lieu of a Pedestrian Railing. At locations where a Combination Railing is used at the exterior edge of a raised sidewalk, a Traffic Railing may be used as an alternative as long as the requirements for Pedestrian Railings are met.



Traffic Railing
All Design Speeds



Combination Railling
Design Speeds of 45 mph or Less



Pedestrian Railing
All Design Speeds

Figure 30.2-1
Bridge Railing Types

The application of bridge railings shall comply with the following guidance:

1. All bridge railings shall conform to **LRFD [13]**.

2. Traffic Railings placed on state-owned and maintained structures (Interstate Highways, United States Highways, State Trunk Highways) with a design speed exceeding 45 mph shall be solid concrete parapets. Where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints, designer shall utilize open railings as described in this section.

   Traffic Railings placed on locally-owned and maintained structures (County Trunk Highways, Local Roadways) with a design speed exceeding 45 mph are strongly encouraged to utilize solid concrete parapets.

3. Traffic Railings placed on structures with a design speed of 45 mph or less can be either solid concrete parapets or open railings with the exception as noted below in the single slope parapet application section.

4. New bridge plans utilizing concrete parapets shall be designed with "SS" ("32SS", "36SS", "42SS", or "56SS") parapets.

5. Per **LRFD [13.7.3.2]**, the minimum Traffic Railing height shall be 27" for TL-3, 32" for TL-4, 42" for TL-5, and 90" for TL-6. The railing applications as noted in this section meet these requirements.

6. Per **LRFD [13.8.1]** and **LRFD [13.9.2]**, the minimum height of a Pedestrian (and/or bicycle) Railing shall be 42" measured from the top of the walkway or riding surface respectively. Per the *Wisconsin Bicycle Facility Design Handbook*, on bridges that are signed or marked as bikeways and bicyclists are operating right next to the railing, the preferred height of the railing is 54". The higher railing/parapet height is especially important and should be used on long bridges, high bridges, and bridges having high bicyclist volumes. If an open railing is used, the clear opening between horizontal elements shall be 6 inches or less.

7. Aesthetics associated with bridge railings shall follow guidance provided in Section 30.4.

The designation for railing types are shown on the Standard Details. Bridge railings shall be employed as follows:

1. The single slope parapet "32SS" shall be used as a Traffic Railing on all structures with a design speed exceeding 45 mph. The "36SS" and "42SS" parapets should be used where the Region determines that there is high truck traffic and/or curved horizontal alignment creating more potential for overtopping the parapet, or if roadway concrete barrier single slope (CBSS) of the same height adjoins the bridge. Single slope parapet "56SS" shall only be used if 56" CBSS adjoins the bridge. The "SS" parapets were crash-tested per NCHRP Report 350 specifications and meet crash test criteria for TL-4.

A “SS” or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and “HF” parapets shall be used as Traffic Railings for rehabilitation projects only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The “51F” parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. The vertical face parapet “A” can be used for all design speeds. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. Under some circumstances, the vertical face parapet “A” can be used as a Traffic Railing for design speeds exceeding 45 mph with the approval of the Bureau of Structures Development Section. The vertical face parapet “A” was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic railings may be used if crash tested according to Section 30.1 or follow the guidance provided in Section 30.4. See Chapter 4 – Aesthetics for CSS considerations.

   The Texas style aesthetic parapet, type “TX”, can be used as a Traffic/Pedestrian Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type “TX” parapet can be used. This parapet is very expensive; however, form liners simulating the openings can be used to reduce the cost of this parapet. The type “TX” parapet was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. The type “PF” tubular railing, as shown in the Standard Details of Chapter 40, shall <u>not</u> be used on new bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type “PF” railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type “C1” through “C6”, are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5” from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. Combination railing, type “3T”, without the recessed details on the parapet faces may be used when aesthetic details are not desired or when CSS funding is not available (see Chapter 4 – Aesthetics). These railings can only be used when the design speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete

parapets to which they are attached (i.e., if a type “C1” combination railing is attached to the top of a vertical face parapet type “A”, the parapet and railing combination meet crash test criteria for TL-4).

8. Chain Link Fence and Tubular Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between the roadway and a sidewalk at grade. Chain Link Fence, when attached to the top of a concrete Traffic Railing, can be used for design speeds exceeding 45 mph. Due to snagging and breakaway potential of the vertical spindles, Tubular Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets (“A” or “SS”) as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: “Type H (insert railing type) railing shall not be used”. The combination railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. Although the type “W” railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 (based on a May 1997 FHWA memorandum), FHWA has since restricted its use as indicated above.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is not allowed

for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing was crash-tested per NCHRP Report 350 and meets criteria for TL-4.

13. Type “NY3/NY4” steel railings, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “NY3/NY4” railings may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “NY3/NY4” railings also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. The type “NY4” railing may be used on a raised sidewalk where the design speed is 45 mph or less. However, the type “NY” railings are not allowed for use on prestressed box girder bridges. These railings shall be considered where the Region requests an open railing. The type “NY” railings were crash-tested per NCHRP Report 350 and meet criteria for TL-4.

14. The type "F" steel railing, as shown in the Standard Details of Chapter 40, shall <u>not</u> be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less. Details in Chapter 40 are for <u>informational purposes only</u>.

15. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the *Facilities Development Manual (FDM) Standard Detail Drawings (SDD) 14b20*. Railing is not required on box culverts if the culvert is extended to provide an adequate clear zone as defined in *FDM 11-15-1*. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. The railing shall be provided only when it is cost effective as defined in *FDM Procedure 11-45-1*.

16. When the structure approach thrie beam is extended across the box culvert; refer to Standard Detail, Box Culvert Details for additional information. The minimum dimension between end of box and face of guard rail provides an acceptable rail deflection to prevent a vehicle wheel from traversing over the end of the box culvert. In almost every case, the timber posts with offset blocks and standard beam guard are used. Type "W" railing may be used for maintenance and box culvert extensions to mitigate the effect of structure modifications.

See the *FDM* for additional railing application requirements. See *11-45-1* and *11-45-2* for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See *11-35-1*, *Table 1.2* for requirements when barrier wall separation between roadway and sidewalk is necessary.

30.3 General Design Details

1. Epoxy coated bars are required for all concrete parapets, curbs, medians, and sidewalks.

2. Adhesive anchored parapets are allowed at interior Traffic Railing locations only when the adjacent exterior parapet is a crash test approved Traffic Railing per Section 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standard Details 30.10 and 30.14 for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in *FDM 11-45 Section 2.3.6.2.2* and *Section 2.3.6.2.3* respectively.

4. It is desirable to avoid attaching noise walls to bridge railings. However, in the event that noise walls are required to be located on bridge railings, compliance with the setback requirements stated in Section 30.4 and what is required in *FDM 11-45 Sections 2.3.6.2.2* and *2.3.6.2.3* is not required. *Note: WisDOT is currently investigating the future use of noise walls on bridge structures in Wisconsin.*

5. Temporary bridge barriers shall be designed in accordance with *FDM SDD 14b7*. Where temporary bridge barriers are being used for staged construction, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

6. Provide for expansion movement in tubular railings where expansion devices or concrete parapet deflection joints exist on the structure plan details. The tubular railing splice should be located over the joint and spaced evenly between railing posts. The tubular railing splice should be made continuous with a movable internal sleeve. If tubular railing is employed on conventional structures where expansion joints are likely to occur at the abutments only, the posts may be placed at equal spacing provided that no post is nearer than 2 feet from deflection joints in the parapet at the piers.

7. Refer to Standard Detail 30.07 – Vertical Face Parapet “A” – for detailing concrete parapet or sidewalk deflection joints. These joints are used based on previous experience with transverse deck cracking beneath the parapet joints.

8. Horizontal cracking has occurred in the past near the top of some concrete parapets which were slip formed. Similar cracking has not occurred on parapets cast in forms. Therefore, slip forming of bridge parapets shall not be allowed.

9. For beam guard type “W” railing, locate the expansion splice at a post or on either side of the expansion joint.

10. Sidewalks - If there is a Traffic Railing between the roadway and an at grade sidewalk, and the roadway side of the Traffic Railing is more than 11’-0” from the exterior edge of deck, access must be provided to the at grade sidewalk for the snooper truck to inspect the underside of the bridge. The sidewalk width must be 10’-0” clear between barriers, including fence (i.e., use a straight fence without a bend). For protective

screening, the total height of parapet and fence need not exceed 8’-0”. The boom extension on most snooper trucks does not exceed 11’-0” so provision must be made to get the truck closer to the edge.

11. Where Traffic Railing is utilized between the roadway and an at grade sidewalk, early coordination with the roadway designer should occur to provide adequate clearances off of the structure to allow for proper safety hardware placement and sidewalk width. Additional clearance may be required in order to provide a crash cushion or other device to protect vehicles from the blunt end of the interior Traffic Railing off of the structure.

12. On shared-use bridges, fencing height and geometry shall be coordinated with the Region and the DNR (or other agencies) as applicable. Consideration shall be given to bridge use (i.e., multi-use/snowmobile may require vertical and horizontal clearances to allow grooming machine passage) and location (i.e., stream crossing vs. grade separation).

13. Per **LRFD [13.7.1.1]**, the use of raised sidewalks on structures shall be restricted to roadways with a design speed of 45 mph or less. The height of curbs for sidewalks is usually 6 inches. This height is more desirable than higher heights with regards to safety because it is less likely to vault vehicles. However, a raised curb is not considered part of the safety barrier system. On structure rehabilitations, the height of sidewalk may increase up to 8 inches to match the existing sidewalk height at the bridge approaches. Contact the Bureau of Structures Development Section if sidewalk heights in excess of 8 inches are desired. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for typical raised sidewalk detail information.

14. Pedestrian loads, as described in the AASHTO LRFD Bridge Design Specifications, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

30.4 Railing Aesthetics

Railing aesthetics have become a key component to the design and delivery of bridge projects in Wisconsin. WisDOT Regions, local communities and their leaders use rail aesthetics to draw pedestrians to use the walkways on structures. With the increased desire to use, and frequency of use of aesthetics on railings, it has become increasingly important to set policy for railing aesthetics on bridge structures.

Railing aesthetics policies have been around for multiple decades. In the 1989 version of the AASHTO Standard Specifications, generalities were listed for use with designing bridge rails. Statements such as "Use smooth continuous barrier faces on the traffic side" and "Rail ends, posts, and sharp changes in the geometry of the railing shall be avoided to protect traffic from direction collision with the bridge rail ends" were used as policy and engineering judgment was required by each individual designer. This edition of the Standard Specifications aligned with NCHRP Report 350.

Caltrans conducted full-scale crash testing of various textured barriers in 2002. This testing was the first of its kind and produced acceptable railing aesthetics guidelines for single slope barriers for NCHRP Report 350 TL-3 conditions. Some of the allowable aesthetics were: sandblast textures with a maximum relief of 3/8", geometric patterns inset into the face of the barrier 1" or less and featuring 45° or flatter chamfered or beveled edges, and any pattern or texture with a maximum relief of 2½" located 24" above the base of the barrier. Later in 2002, Harry W. Taylor, the Acting Director of the Office of Safety Design of FHWA, provided a letter to Caltrans stating that their recommendations were acceptable for use on all structure types.

In 2003, WisDOT published a paper titled, "Acceptable Community Sensitive Design Bridge Rails for Low Speed Streets & Highways in Wisconsin". The goal of this paper was to streamline what railing aesthetics were acceptable for use on structures in Wisconsin. WisDOT policy at that time allowed vertical faced bridge rails in low speed applications to contain aesthetic modifications. For NHS structures, WisDOT allowed various types of texturing and relief based on crash testing and analysis. Ultimately, WisDOT followed many of the same requirements that were deemed acceptable by FHWA based on the Caltrans study in 2002.

NCHRP Report 554 – Aesthetic Concrete Barrier Design – was published in 2006 to (1) assemble a collection of examples of longitudinal traffic barriers exhibiting aesthetic characteristics, (2) develop design guidelines for aesthetic concrete roadway barriers, and (3) develop specific designs for see-through bridge rails. This publication serves as the latest design guide for aesthetic bridge barrier design and all bridge railings on structures in Wisconsin shall comply with the guidance therein.

<u>The application of aesthetics on bridge railings on structures in Wisconsin with a design speed exceeding 45 mph shall comply with the following guidance</u>:

1. All Traffic Railings shall meet the crash testing guidelines outlined in Section 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt

ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed as follows:

Minimum of 2'-3" behind the front face toe of the parapet when used with single slope parapets ("32SS", "36SS", "42SS", or "56SS").

Minimum of 2'-6" behind the front face toe of the parapet when used with sloped face parapets ("LF" or "HF").

Minimum of 2'-0" behind the front face of the parapet when used with vertical face parapets ("A").

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

4. Any concrete parapet placed directly on the deck may contain patterns or textures of any shape and length inset into the front face with the exception noted in #5. The maximum pattern or texture recess into the face of the barrier shall be ½". Note that the typical aesthetic form liner patterns shown in Standard Detail 4.01 are not acceptable for use on the front face of vehicle barriers.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings; especially in high speed applications where the aesthetic features will be negligible to the traveling public. In addition to the increased risk of vehicle snagging, aesthetic treatments on the front face of traffic railings are exposed to vehicle impacts, snowplow scrapes, and exposure to deicing chemicals. Due to these increased risks, future maintenance costs will increase.

5. No patterns with a repeating upward sloping edge or rim in the direction of vehicle traffic shall be permitted.

6. Staining should not be applied to the roadway side face of concrete traffic railings.

The application of aesthetics on bridge railings on structures in Wisconsin with a roadway design speed of 45 mph or less shall comply with the following guidance (see Chapter 4 – Aesthetics for CSS funding implications):

1. All Traffic Railings shall meet the crash testing guidelines outlined in Section 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed a minimum of 1'-0" behind the front face toe of the parapet.

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

4. Any concrete parapet placed directly on the deck or Combination Railing on a raised sidewalk may contain geometric patterns inset into the front face. The maximum recess

into the face of the barrier shall be 1” and shall be placed concurrently with a 45° or flatter chamfered or beveled edge. See Standard Details 30.17 and 30.18 for one example of this type of aesthetic modification.

WisDOT highly recommends the use of smooth front faces of Traffic Railings and Combination Railings.

5. Any concrete parapet placed directly on the deck or Combination Railing on a raised sidewalk may contain textures of any shape and length inset into the front face. The maximum depth of the texture shall be ½”. Note that the typical aesthetic form liner patterns shown in Standard Detail 4.01 are not acceptable for use on the front face of vehicle barriers.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings and Combination Railings.

6. No patterns with a repeating upward sloping edge or rim in the direction of vehicle traffic shall be permitted.

7. Staining should not be applied to the roadway side face of concrete traffic railings. Staining is allowed on concrete surfaces of Combination Railings placed on a raised sidewalk.

30.5 Utilities

The maximum allowable conduits that can be placed in "SS", "LF" or "HF" parapets are shown in the following sketches ("32SS" shown). Junction (Pull) boxes can only be used with 2 inch diameter conduit. The maximum length of 3 inch conduit is 190 feet, as no boxes are allowed.

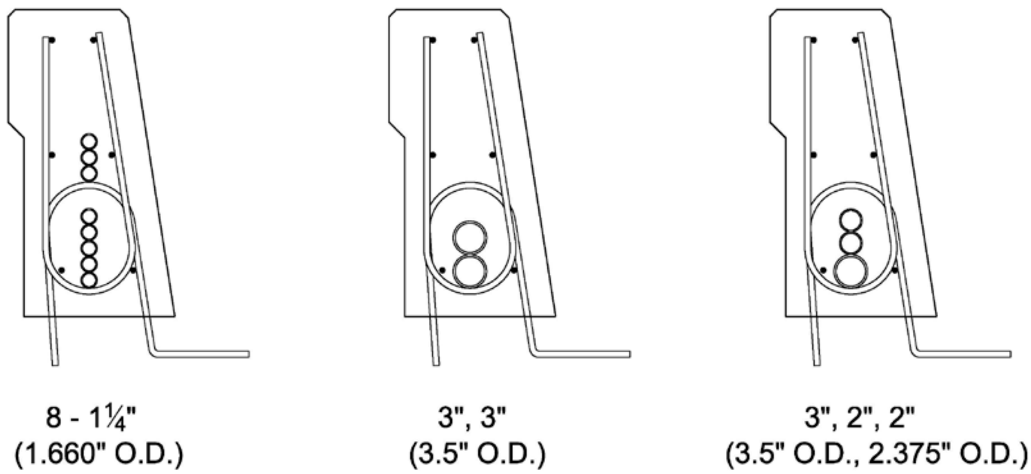


Figure 30.5-1
Maximum Allowable Conduits in "SS", "LF" and "HF" Parapets

When light poles are mounted on top of parapets and the design speed exceeds 45 mph, the light pole must be located behind the back edge of the parapet. See Standard Detail 30.21 – Light Standard, Junction Box, & Expansion Fitting for "SS" Parapets – for typical light pole blister detail information. The poles should also be placed over the piers unless there is an expansion joint at that location. If an expansion joint is present, place 4 feet away. *FDM 9-25-5* addresses whether a bench mark disk should be set on a structure; however, structures are not usually preferred due to possible elevation changes from various causes. See Bridge Manual section 6.3.3.7 for more information regarding bench mark disks.

30.6 Protective Screening

Protective screening is a special type of fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a Traffic Railing (part of a Combination Railing) or on a sidewalk surface (Pedestrian Railing). The top of the protective screening may be bent inward toward the roadway, if mounted on a Traffic Railing and on a raised sidewalk, to prevent objects from being thrown off the overpass structure. The top of the protective screening may also be bent inward toward the sidewalk, if mounted directly to the deck when it is protected by a Traffic Railing between the roadway and a sidewalk at grade. Aesthetics are enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 30 and Chapter 37 Standard Details for protective screening detail information.

Examples of situations that warrant consideration of protective screening are:

1. Location with a history of, or instances of, objects being dropped or thrown from an existing overpass.

2. All new overpasses if there have been instances of objects being dropped or thrown at other existing overpasses in the area.

3. Overpasses near schools, playgrounds, residential areas or any other locations where the overpass may be used by children who are not accompanied by an adult.

In addition, all pedestrian overpasses should have protective screening on both sides.

Protective screening is not always warranted. An example of when it may not be warranted is on an overpass without sidewalks where pedestrians do not have safe or convenient access to either side because of high traffic volumes and/or the number of traffic lanes that must be crossed.

When protective screening is warranted, the minimum design should require screening on the side of the structure with sidewalk. Designers can call for protective screening on sides without sidewalks if those sides are readily accessible to pedestrians.

Designers should ensure that where protective screening is called for, it does not interfere with sight distances between the overpass and any ramps connecting it with the road below. This is especially important on cloverleaf and partial cloverleaf type interchanges.

Protective screening (or Pedestrian Railing) may be required for particular structures based on the safety requirements of the users on the structure and those below. Roadway designers, bridge designers, and project managers should coordinate this need and relay the information to communities involved when aesthetic details are being formalized.

See *FDM 11-35-1.8* for additional guidance pertaining to protective screening usage requirements.

Occasionally, access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

Fence repair should follow this same process except the damaged fencing would be removed and replaced with new fencing.

See Section 30.3 for additional guidance with regards to snooper truck access, screening height, and straight vs. bent fencing.

30.7 Medians

The typical height of any required median curb is 6 inches. This will prevent normal crossovers and reduce vaulting on low speed roadways without excessive dead load being applied to the superstructure. On structure rehabilitations, the height of median may increase up to 8” to match the existing median at the bridge approaches. Contact the Bureau of Structures Development Section if median heights in excess of 8 inches are desired. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for typical raised median detail information.

30.8 Railing Rehabilitation

The FHWA, in its implementation plan for MASH, requires that bridge railings on the NHS shall meet the requirements of MASH or NCHRP Report 350. In addition, FHWA states that "Agencies are encouraged to upgrade existing highway safety hardware that has not been accepted under MASH or NCHRP Report 350 during reconstruction projects, during 3R (Resurfacing, Restoration, Rehabilitation), or when the railing system is damaged beyond repair".

WisDOT requirements for the treatment of existing railings for various project classifications are outlined in Table 30.8-1:

Project Classification	Railing Rehabilitation
Preventative Maintenance✳ (Resurfacing, Restoration) For letting dates after December 31, 2019: The compliance document will be MASH 2016 Edition	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required. Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350. Non-NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350.

3R✳✳ (Resurfacing, Restoration, Rehabilitation) For letting dates after December 31, 2019: The compliance document will be MASH 2016 Edition	If rehabilitation work, as part of the 3R project, is scheduled or performed which does not widen the structure nor affect the existing railing.	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required provided the minimum rail height requirement is met. (Minimum rail height shall be 27" for roadway design speed of 45 mph or less and 32" for roadway design speed exceeding 45 mph.) Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings (i.e., raised to meet the minimum rail height requirement) where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. NHS Structures: Existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement shall be upgraded to comply with MASH or NCHRP Report 350. Non-NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement be upgraded to comply with MASH or NCHRP Report 350.
	If rehabilitation work, as part of the 3R project, is scheduled or performed which widens the structure to either side, redecks (full-depth) any complete span of the structure, or if any work affecting the rail is done to the existing structure.	All railing on the structure must comply with MASH or NCHRP Report 350. Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.
4R (Resurfacing, Restoration, Rehabilitation, Reconstruction) For letting dates after December 31, 2019: The compliance document will be MASH 2016 Edition		All railing on the structure must comply with MASH or NCHRP Report 350. Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.

Table 30.8-1

WisDOT Requirements for Retrofitting/Upgrading Bridge Railings to Current Standards

✳ Examples of Preventative Maintenance projects include, but are not limited to:

1. Bridge deck work: Concrete deck repair, patching, and concrete overlays; asphaltic overlays; epoxy and polymer overlays; expansion joint replacement when done in conjunction with an overlay or expansion joint elimination; chloride extraction; installation of a cathodic protection system.

2. Superstructure and substructure work: Steel structure cleaning and repainting, including complete repainting, zone painting, and spot painting with overcoat; structural repairs (except vehicle impact damage); bearing repair or replacement.

✳✳ Examples of 3R projects include, but are not limited to:

1. Bridge deck work: Bridge deck widenings and re-decks; expansion joint replacement when done in conjunction with an overlay or expansion joint elimination; approach slab replacement.

2. Superstructure and substructure work: Wing wall replacement; emergency bridge repair; structural repairs to railings based on vehicle impact damage;

The minimum railing height shall be measured from the top inside face of the railing to the top of the roadway surface at the toe of railing.

For all railing rehabilitations that require upgrades to comply with MASH or NCHRP Report 350, railings shall be employed as discussed in Section 30.2.

The following is a list of typical railing types that are in service on structures in Wisconsin. The <u>underlined</u> railings comply with MASH, NCHRP Report 350, or NCHRP Report 230 and may remain in service within rehabilitation projects. The *italicized* railings do not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and shall be removed from service within rehabilitation projects.

1. <u>Single slope parapet “32SS”, “36SS”, “42SS”, “56SS”</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 specifications and meet crash test criteria for TL-4.

2. <u>Sloped face parapet "LF"</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. <u>Sloped face parapet "HF"</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

4. <u>Vertical face parapet “A”</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. <u>Aesthetic parapet “TX”</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. Type “PF” tubular railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum. Standard Details in Chapter 40.

7. Type “H” railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

8. Timber Railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

9. Type "W" railing. Railing may be used for rehabilitation projects on non-NHS structures only. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 based on a May 1997 FHWA memorandum.

10. Type “M” railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 and meets TL-4

11. *Type “F” railing.* Railing may not be used for rehabilitation projects. Standard Details in Chapter 40 are for informational purposes only.

12. Sloped face parapet “B”. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230. Standard Details in Chapter 40.

The region shall contact the Bureau of Structures Development Section to determine the sufficiency of existing railings not listed above.

Rehabilitation or improvement projects to historically significant bridges require special attention. Typically, if the original railing is present on a historic bridge, it will likely not meet current crash testing requirements. In some cases, the original railing will not meet current minimum height and opening requirements. There are generally two different options for upgrading railings on historically significant bridges – install a crash-tested Traffic Railing to the interior side of the existing railing and leave the existing railing in place or replace the existing railing with a crash-tested Traffic Railing. Other alternatives may be available but consultation with the Bureau of Structures Development Section is required.

30.9 Railing Guidance for Railroad Structures

Per an April 2013 memorandum written by M. Myint Lwin, Director of the FHWA Office of Bridge Technology, bridge parapets, railings, and fencing shall conform to the following requirements when used in the design and construction of grade separated highway structures over railroads:

1. For NHS bridges over railroad:

   Bridge railings shall comply with AASHTO standards. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   However, railings for use on NHS bridges over railroads shall be governed by the railroad's standards, regardless of whether the bridge is owned by the railroad or WisDOT. For the case where an NHS bridge crosses over railroads operated by multiple authorities with conflicting parapet, railing, or fencing requirements, standards as agreed by the various railroad authorities and as approved by WisDOT shall be used.

2. For non-NHS bridges over railroad:

   Bridge railings shall comply with the policies outlined within this chapter. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   All federally funded non-NHS bridges including those over railroads shall be governed by WisDOT's policies outlined above, even if they differ from the railroad's standards.

30.10 References

1. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications.*

2. American Association of State Highway and Transportation Officials. *Manual for Assessing Safety Hardware*.

3. National Cooperative Highway Research Program. *NCHRP Report 554 – Aesthetic Concrete Barrier Design*.

4. State of California, Department of Transportation. *Crash Testing of Various Textured Barriers*.

5. National Cooperative Highway Research Program. *NCHRP Report 350 – Recommended Procedures for the Safety Performance Evaluation of Highway Features*.

6. State of Wisconsin, Department of Transportation. *Facilities Development Manual*.

7. State of Wisconsin, Department of Transportation. *Wisconsin Bicycle Facility Design Handbook.*

8. State of Wisconsin, Department of Transportation. *Memorandum of Understanding between Wisconsin Department of Transportation, Wisconsin County Highway Association, and Transportation Builders Association.*

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32.1 General

The Regional Office shall determine the utilities that will be affected by the construction of any bridge structure at the earliest possible stage. It shall be their responsibility to deal with these utilities and to provide location plans or any other required sketches for their information. When the utility has to be accommodated on the structure, the Regional Office shall secure approval from the representative of the utility and the Bureau of Structures for the location and method of support.

Due consideration shall be given to the weight of the pipes, ducts, etc. in the design of the beams and diaphragms. To insure that the function, aesthetics, painting and inspection of stringers of a structure are maintained, the following applies to the installation of utilities on structures:

1. Permanent installations, which are to be carried on and parallel to the longitudinal axis of the structure, shall be placed out of sight, between the fascia beams and above the bottom flanges, on the underside of the structure.

2. Conglomeration of utilities in the same bay shall be avoided in order to facilitate maintenance painting and future inspection of girders in a practical manner.

3. In those instances where the proposed type of superstructure is not adaptable to carrying utilities in an out-of-sight location in the underside of the structure, an early determination must be made as to whether or not utilities are to be accommodated and, if so, the type of superstructure must be selected accordingly.

32.2 Plans

Utilities may be supported by a system which requires inserts in the concrete deck slab. They also may be supported directly on structural beams. Utilities shall not be supported by a system that requires drilling into prestressed concrete beams or welding onto structural steel beams.

It shall be the responsibility of the Regional Office to obtain approval of support details from the individual utility companies prior to the final submission.

Preliminary and final general plan and elevation drawings shall show information about all existing and proposed utilities carried under the superstructure or in the vicinity of foundations. Complete information as to the name of owner, size, type, abandonment, proposed relocation, material to be furnished by utility company, etc. shall be noted.

32.3 Department Policy

The following guidance in regard to utility installations on bridges should be followed:

General Considerations

1. In most cases, attachment of utility facilities to highway structures, such as bridges, is a practical arrangement and considered to be in the public interest. However, attaching utility lines to a highway structure can materially affect the structure, the safe operation of traffic, the efficiency of maintenance as well as the appearance and therefore must be provided for during the design stage.

2. Since highway structure designs and site conditions vary, the adoption of a standard method to accommodate utility facilities is not feasible; however, the method employed should conform to logical engineering considerations for preserving the highway, its safe operation, maintenance and appearance. Generally, acceptable utility installations are those which will occupy a position beneath the structure's floor, between the outer girders of beams or within a cell, and at an elevation above low superstructure steel or masonry.

3. The general controls for providing encasement, allied mechanical protection and shut-off valves to pipeline crossings of highways and for restriction against varied use shall be followed for pipeline attachments to bridge structures, except that sleeves are required only through the abutment backwalls. Where a pipeline attachment to a bridge is encased, the casing should be effectively opened or vented at each end to prevent possible buildup of pressure and to detect leakage of gases or fluid.

   Since an encasement is not normally provided for a pipeline attachment to a bridge, additional protective measures shall be taken. Such measures shall employ higher factor of safety in the design, construction, and testing of the pipeline than would normally be required for cased construction.

4. Communication and electric power line attachments shall be suitably insulated, grounded, and carried in protective conduit or pipe from the point of exit from the ground to re-entry. The cable shall be carried to a manhole located beyond the backwall of the structure. Carrier pipe and casing pipe should be suitably insulated from electric power line attachments.

5. Guy wires in support of any utility will never be allowed to attach to a bridge structure.

6. Cell phone or other type antennas shall not be mounted from or on any bridge or sign support structure.

32.4 Pipeline Expansion Joints

Allowances must be made for changes in pipe length due to thermal expansion and alternate contraction. While couplings will take care of the normal amount of expansion and contraction in each length of pipe, expansion joints are required where no flexible joints are used in the pipeline or when the amount of concentrated movement at one point in excess of the amount that can be safely absorbed by the standard coupling.

An expansion joint should be located in the pipeline adjacent to every point where expansion means are provided in the superstructure.

Use couplings to accommodate the differential movement between the structure and the line itself, and to provide flexibility to accommodate vibrations of the structure. Each coupling can safely accommodate up to 3/8 inch longitudinal movement.

Proper alignment is important to insure free and concentric movement of the slip-type expansion joint. Alignment guides should be designed to allow free movement in only one direction along the axis of the pipe and to prevent any horizontal or vertical movement of the pipe. Suitable pipe alignment guides may be obtained from reliable pipe alignment guide manufacturers. Alignment guides should be fastened to some rigid part of the installation, such as the framework of the bridge. Alignment guides should be located as close to the expansion joint as possible, up to a maximum of 4 pipe diameters. The distance from the first pipe guide to the second should not exceed a maximum of 14 pipe diameters from the first guide. Where an anchor is located adjacent to an expansion joint, it too, should be located as close to the expansion joint as possible – to a maximum of 4 pipe diameters from the expansion-joint. Additional pipe supports are usually required. Pipe supports should not be substituted for alignment guides.

The main pipe anchors must be designed to withstand the full thrust resulting from internal line pressure; also, the force required to collapse the slip pipe, and the frictional forces due to guides and supports.

32.5 Lighting

When lighting conduits are used in a bridge use an approved expansion fitting at each semi expansion or expansion joint.

Use bolted option on all bridges with X-frame and lower laterals. Do not use bolted option when channel diaphragms are used.

There is some flexibility in placing light standards. Whenever possible, place all light standards at the piers instead of in the spans for both aesthetics and vibration problems. Place 4' from pier if there is an expansion joint at the pier. WisDOT has experienced mast arm failures due to vibration on poles placed further from the pier.

With poles set in the center of the spans on bridges the heavy luminaire tends to stand still as the bridge deflects due to traffic. The pole shaft is too stiff to deflect much so the pole arm takes all the movement.

With a constant wind velocity the poles will vibrate. If they are placed too far into the span, deflections from traffic will induce further erratic vibrations. While single arm brackets are aesthetically appealing they are more prone to fatigue failures than the double arm brackets. Some single arm brackets have been replaced this way.

The resonant frequency of most poles is quite low (5 to 10 cycles per second). Therefore low wind velocities can excite these poles if they are not damped. In most cases the arm and luminaire do some of this. One case where this didn't work was corrected by putting vermiculite in the pole.

Some pole vibrations cause the bulbs to unscrew and fall out. This is corrected by attaching a clamp over the end of the bulb.

55 foot long poles with 20 foot mast arms can have a noticeable bend in the pole due to the dead load of the luminaire and mast arm up to approximately 12 inches.

The pole manufacturers suggest that the poles be manufactured with a curve so that the dead load of the arm and luminaire cause the centerline of the pole to approximate a straight line. They did not want to increase the pole cost by using more material. A fair tolerance should be allowed on the prescribed shape of the pole.

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36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Currently, the *Bureau of Structures* does not require rating calculations for box culverts. See 45.8 for values to place on the plans for inventory and operating rating factors.

WisDOT Policy Item:

Current WisDOT policy is to not rate box culverts. In the future, rating requirements will be introduced as *AASHTO Manual for Bridge Evaluation (LRFR)* is updated to more thoroughly address box culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.6 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans. The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* does not thoroughly cover rating of concrete box culverts. See 45.8 for values to place on the plans for maximum (Wis-SPV) vehicle load.

36.2 General

Box culverts are reinforced concrete closed rigid frames which must support vertical earth and truck loads and lateral earth pressure. They may be either single or multi-cell. The most common usage is to carry water under roadways, but they are frequently used for pedestrian or cattle underpasses.

Box culverts used to carry water should consider the following items:

- Hydraulic and other requirements at the site determine the required height and area of the box. Hydraulic design of box culverts is described in Chapter 8.
- Once the required height and area is determined, the selection of a single or multi-cell box is determined entirely from economics. Barrel lengths are computed to the nearest 6 inches. For multi-cell culverts the cell widths are kept equal.
- A minimum vertical opening of 5 feet is desirable for cleaning purposes.

Pedestrian underpasses should consider the following items:

- The minimum opening for pedestrian underpasses is 8 feet high by 10 feet wide. However, when considering maintenance and emergency vehicles or bicyclists the minimum opening should be 10 feet high by 12 feet wide. For additional guidance refer to the Wisconsin Bicycle Facility Design Handbook and the FDM.
- The entire top and 1 foot below the bottom of the top slab should be waterproofed.
- The top of the bottom slab should be sloped with a 1% normal crown to minimize moisture collecting on the travel path. Additionally, 0.5% to 1% longitudinal slope for drainage is recommended.
- Flared wings are recommended at openings. For long underpasses, lighting systems (recessed lights and skylights) should be considered, as well. For additional guidance on user’s comfort, safety measures, and lighting refer to the Wisconsin Bicycle Facility Design Handbook.

Cattle underpasses should consider the following items:

- The minimum size for cattle underpasses is 6 feet high by 5 feet wide.
- Consider providing a minimum longitudinal slope of 1%, desirable 3%, to allow for flushing, but not so steep that the stock will slip. Slopes steeper than 5% should be avoided.
- For additional guidance refer to the FDM.

Aluminum box culverts are not permitted by the Bureau of Structures.

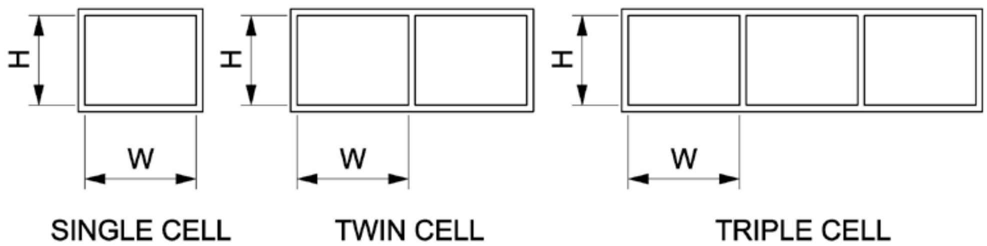


Figure 36.2-1
Typical Cross Sections

36.2.1 Material Properties

The properties of materials used for concrete box culverts are as follows:

f'_c	=	specified compressive strength of concrete at 28 days, based on cylinder tests
	=	3.5 ksi for concrete in box culverts
f_y	=	60 ksi, specified minimum yield strength of reinforcement (Grade 60)
E_s	=	29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**
E_c	=	modulus of elasticity of concrete in box **LRFD [C5.4.2.4]**
	=	$(33{,}000)(K_1)(w_C)^{1.5}(f'_C)^{1/2}$ = 3586 ksi

Where:

K_1	=	1.0
W_C	=	0.15 kcf, unit weight of concrete
n	=	Es / Ec = 8, modular ratio **LRFD [5.7.1]**

36.2.2 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Consider the hydraulics of the highway-stream crossing system in choosing the preferred design from the available alternatives. Estimates of life cycle costs and risks associated with each alternative help indicate which structure to select. Consider construction costs, maintenance costs, and risks of future costs to repair flood damage. Other considerations which may influence structure-type selection are listed in Table 36.2-1.

Bridges	
Advantages	Disadvantages
Less susceptible to clogging with drift, ice and debris	Require more structural maintenance than culverts
Waterway width increases with rising water surface until water begins to submerge structure	Piers and abutments susceptible to scour failure
Natural bottom for waterway	Susceptible to ice and frost forming on deck
Culverts	
Grade rises and widening projects sometimes can be accommodated by extending culvert ends	Silting in multiple barrel culverts may require periodic cleanout
Minimum structural maintenance	No increase in waterway area as stage rises above top of culvert
Usually easier and quicker to build than bridges	May clog with drift, debris or ice

Table 36.2-1
Advantages/Disadvantages of Structure Type

36.2.3 Staged Construction for Box Culverts

The inconvenience to the traveling public has often led to staged construction projects. Box culverts typically work well with staged construction. Any cell joint can be used for a staging joint. When the construction staging line cannot be determined in design to locate a cell joint, a contractor placed construction joint can be done with an extra set of dowel bars and the contractor field cutting the longitudinal bars.

36.3 Limit States Design Method

36.3.1 LRFD Requirements

For box culvert design, the component dimensions and the size and spacing of reinforcement shall be selected to satisfy the following equation for all appropriate limit states, as presented in **LRFD [1.3.2.1]:**

$$Q = \sum \eta_i \, \gamma_i \, Q_i \leq \phi R_n = R_r$$

Where:

η_i	=	Load modifier (a function of η_D, η_R, and η_I)
γ_i	=	Load factor
Q_i	=	Force effect: moment, shear, stress range or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance: resistance of a component to force effects
R_r	=	Factored resistance = ϕR_n

See 17.2.2 for load modifier values.

36.3.2 Limit States

The Strength I Limit State is used to design reinforcement for flexure and checking shear in the slabs and walls, **LRFD [12.5.3]**. The Service I Limit State is used for checking reinforcement for crack control criteria, **LRFD [12.5.2]**, and checking settlement of the box culvert as shown in 36.8.1.

Per **LRFD [C12.5.3, 5.5.3],** buried structures have been shown not to be controlled by fatigue.

WisDOT Policy Item:

Fatigue criteria are not required on any reinforced concrete box culverts, with or without fill on the top slab of the culvert. This policy item is based on the technical paper titled "Fatigue Evaluation for Reinforced Concrete Box Culverts" by H Hany Maximos, Ece Erdogmus, and Maher Tadros, published in the ACI Structural Journal, January/February 2010.

36.3.3 Load Factors

In accordance with **LRFD [Table 3.4.1-1 and Table 3.4.1-2]**, the following Strength I load factors, γ_{st}, and Service I load factors, γ_{s1}, shall be used for box culvert design:

		Strength I Load Factor, γ_{st}		Service I Load Factor, γ_{s1}
Type of Load		Max.	Min.	
Dead Load-Components	DC	1.25	0.90	1.0
Dead Load-Wearing Surface	DW	1.50	0.65	1.0
Vertical Earth Pressure	EV	1.35	0.90	1.0
Horizontal Earth Pressure	EH	1.50	0.50[1]	1.0
Live Load Surcharge	LS	1.75	1.75	1.0
Live Load + IM	LL+IM	1.75	1.75	1.0

[1]Per **LRFD [3.11.7]**, for culverts where earth pressure may reduce effects caused by other loads, a 50% reduction may be used, but not combined with the minimum load factor specified in **LRFD [Table 3.4.1-2]**.

36.3.4 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a structure is expected to experience during its design life **LRFD [1.3.2.4]**.

36.3.4.1 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for the variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for reinforced concrete box culverts for the Strength Limit State per **LRFD [Table 12.5.5-1]** are as shown below:

Structure Type	Flexure	Shear
Cast-In-Place	0.90	0.85
Precast	1.00	0.90
Three-Sided	0.95	0.90

36.3.4.2 Moment Capacity

For rectangular sections, the nominal moment resistance, M_n, per **LRFD [5.7.3.2.3]** (tension reinforcement only) equals:

$$M_n = A_s f_s \left(d_s - \frac{a}{2}\right)$$

The factored resistance, M_r, or moment capacity per **LRFD [5.7.3.2.1]**, shall be taken as:

$$M_r = \phi M_n = \phi A_s f_s \left(d_s - \frac{a}{2}\right)$$

For additional information on concrete moment capacity, including stress and strain assumptions used, refer to 18.3.3.2.1.

The location of the design moment will consider the haunch dimensions in accordance with **LRFD [12.11.4.2]**. No portion of the haunch shall be considered in adding to the effective depth of the section.

36.3.4.3 Shear Capacity

Per **LRFD [12.11.4.1]**, shear in culverts shall be investigated in conformance with **LRFD [5.14.5.3]**. The location of the critical section for shear for culverts with haunches shall be determined in conformance with **LRFD [C5.13.3.6.1]** and shall be taken at a distance d_v from the end of the haunch.

36.3.4.3.1 Depth of Fill Greater than or Equal to 2.0 ft.

The shear resistance of the concrete, V_c, for slabs of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.14.5.3]** shall be determined as:

$$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \leq 0.126\lambda\sqrt{f'_c}\,bd_e$$

Where:

$$\frac{V_u d_e}{M_u} \leq 1$$

Where:

V_c	=	Shear resistance of the concrete (kip)
A_s	=	Area of reinforcing steel in the design width (in^2)

d_e = Effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

V_u = Factored shear (kip)

M_u = Factored moment, occurring simultaneously with V_u (kip-in)

b = Design width (in.)

λ = Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

In the absence of shear reinforcing, the nominal shear resistance is equal to the shear resistance of the concrete. The factored resistance, V_r, or shear capacity, per **LRFD [5.8.2.1]** shall be taken as:

$$V_r = \phi V_n = \phi V_c$$

Per **LRFD [5.14.5.3]**, for single-cell box culverts only, Vc for slabs monolithic with walls need not be taken less than:

$$0.0948 \cdot \lambda \sqrt{f'_c}\, b d_e$$

and V_c for slabs simply supported need not be taken less than:

$$0.0791 \cdot \lambda \sqrt{f'_c}\, b d_e$$

The shear resistance of the concrete, V_c, for walls of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.8.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta \lambda \sqrt{f'_c}\, b_v d_v \leq 0.25 f'_c\, b_v d_v$$

Where:

V_c = Shear resistance of the concrete (kip)

β = 2.0 (**LRFD [5.8.3.4.1]**)

b_v = Effective web width taken as the minimum web width within the depth d_v (in.)

d_v = Effective shear depth as determined in **LRFD [5.8.2.9]**. Perpendicular distance between tension and compression resultants. Need not be taken less than the greater of $0.9d_e$ or $0.72h$ (in.)

λ = Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

36.3.4.3.2 Depth of Fill Less than 2.0 ft

Per **LRFD [5.14.5.3]**, for box culverts with less than 2.0 feet of fill follow **LRFD [5.8]** and **LRFD [5.13.3.6]**.

The shear resistance of the concrete, V_c, for slabs and walls of box culverts with less than 2.0 feet of fill, for one-way action per **LRFD [5.8.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta\lambda\sqrt{f'_c}\,b_v d_v \le 0.25f'_c\,b_v d_v$$

With variables defined above in 36.3.4.3.1.

For box culverts where the top slab is an integral part of the wearing surface (depth of fill equal zero) the top slab shall be checked for two-way action, as discussed in 18.3.3.2.2.

36.3.5 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**.

36.3.5.1 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

36.3.5.2 Crack Control Criteria

Per **LRFD [12.11.3]**, the provisions of **LRFD [5.7.3.4]** shall apply to crack width control in box culverts. All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r, specified in **LRFD [5.4.2.6]** for Service I Limit State. The spacing, s, (in inches) of mild steel reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(in.)}$$

Bar spacing, s, need not be less than 5 in. for control of flexural cracking **LRFD [5.7.3.4]** in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

γ_e	=	Exposure factor (1.0 for Class 1 exposure condition, 0.75 for Class 2 exposure condition, see **LRFD [5.7.3.4]** for guidance)
d_c	=	Thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.). For calculation purposes, d_c, need not be taken greater than 2 in. plus the bar radius
f_{ss}	=	Tensile stress in steel reinforcement at the service limit state (ksi) $\leq$ 0.6 f_y
h	=	Overall thickness or depth of the component (in.)

WisDOT Policy Item:

A class 1 exposure factor, γ_e = 1.0, shall be used for all cases for cast-in-place box culverts except for the top steel in the top slab of a box culvert with zero fill, where a class 2 exposure factor, γ_e = 0.75, shall be used.

36.3.6 Minimum Reinforcement Check

Per **LRFD [12.11.4.3]**, the area of reinforcement, A_s, in the box culvert cross-section should be checked for minimum reinforcement requirements per **LRFD [5.7.3.3.2]**.

The area of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, or moment capacity at least equal to the lesser of:

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3 \, (\gamma_1 f_r) \, S = 1.1 \, f_r \, (I_g / c)$; $S = I_g / c$

Where:

γ_1	=	1.6	flexural cracking variability factor
γ_3	=	0.67	ratio of minimum yield strength to ultimate tensile strength; for <u>A615 Grade 60 reinforcement</u>

f_r = $0.24\lambda\sqrt{f'_c}$ Modulus of rupture (ksi) **LRFD [5.4.2.6]**

I_g = Gross moment of inertia (in^4)

c = ½ *effective slab thickness (in.)

M_u = Total factored moment using Strength I Limit State (kip-in)

M_{cr} = Cracking strength moment (kip-in)

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

The factored resistance, M_r or moment capacity, shall be calculated as in 36.3.4.2 and shall satisfy:

$M_r \geq \min(M_{cr} \text{ or } 1.33\ M_u)$

36.3.7 Minimum Spacing of Reinforcement

Per **LRFD [5.10.3.1]**, the clear distance between parallel bars in a layer shall not be less than:

- 1.5 times the nominal diameter of the bars
- 1.5 times the maximum size of the course aggregate
- 1.5 inches

36.3.8 Maximum Spacing of Reinforcement

Per **LRFD [5.10.3.2]**, the spacing of reinforcement in walls and slabs shall not exceed:

- 1.5 times the thickness of the member (3.0 times for temperature and shrinkage)
- 18 inches

36.3.9 Edge Beams

Per **LRFD [12.11.2.1]**, for cast-in-place box culverts, and for precast box culverts with top slabs having span to thickness ratios (s/t) > 18 or segment lengths < 4.0 feet, edge beams shall be provided as specified in **LRFD [4.6.2.1.4]** as follows:

- At ends of culvert runs where wheel loads travel within 24.0 inches from the end of the culvert
- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint

The edge beam provisions are only applicable for culverts with less than 2.0 ft of fill, **LRFD [C12.11.2.1]**.

36.4 Design Loads

36.4.1 Self-Weight (DC)

Include the structure self-weight based on a unit weight of concrete of 0.150 kcf. When there is no fill on the top slab of the culvert, the top slab thickness includes a ½” wearing surface. The weight of the wearing surface is included in the design, but its thickness is not included in the section properties of the top slab. When designing the bottom slab of a culvert do not forget that the weight of the concrete in the bottom slab acts in an opposite direction than the bottom soil pressure and thus reduces the design moments and shears. This load is designated as, DC, dead load of structural components and nonstructural attachments, for application of load factors and limit state combinations.

36.4.2 Future Wearing Surface (DW)

If the fill depth over the culvert is greater than zero, the weight of the future wearing surface shall be taken as zero. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 20 psf. This load is designated as, DW, dead load of wearing surfaces and utilities, for application of load factors and limit state combinations.

36.4.3 Vertical and Horizontal Earth Pressure (EH and EV)

WisDOT Policy Item:

Box Culverts are assumed to be rigid frames. Use Vertical Earth Pressure load factors for rigid frames, in accordance with **LRFD [Table 3.4.1-2]**.

Use Horizontal Earth Pressure load factors for active soil pressure, in accordance with **LRFD [Table 3.4.1-2]**. Using load factors for active soil pressure is a conservative assumption.

The weight of soil above the buried structure is taken as 0.120 kcf. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil. This coefficient of lateral earth pressure is based on an at-rest condition and an effective friction angle of 30º, **LRFD [3.11.5.2]**. The lateral earth pressure is calculated per **LRFD [3.11.5.1]**:

$$p = k_o \gamma_s z$$

Where:

p	=	Lateral earth pressure (ksf)
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)
z	=	Depth below the surface of earth (ft)

WisDOT Policy Item:

For modification of earth loads for soil-structure interaction, embankment installations are always assumed for box culvert design, in accordance with **LRFD [12.11.2.2]**.

Soil-structure interaction for vertical earth loads is computed based on **LRFD [12.11.2.2]**. For embankment installations, the total unfactored earth load is:

$$W_E = F_e \gamma_s B_c H$$

In which:

$$F_e = 1 + 0.20\frac{H}{B_c}$$

Where:

W_E	=	Total unfactored earth load (kip/ft width)
F_e	=	Soil-structure interaction factor for embankment installations (F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section)
γ_s	=	Unit weight of backfill (kcf)
B_c	=	Outside width of culvert, as specified in Figure 36.4-1 (ft)
H	=	Depth of fill from top of culvert to top of pavement (ft)

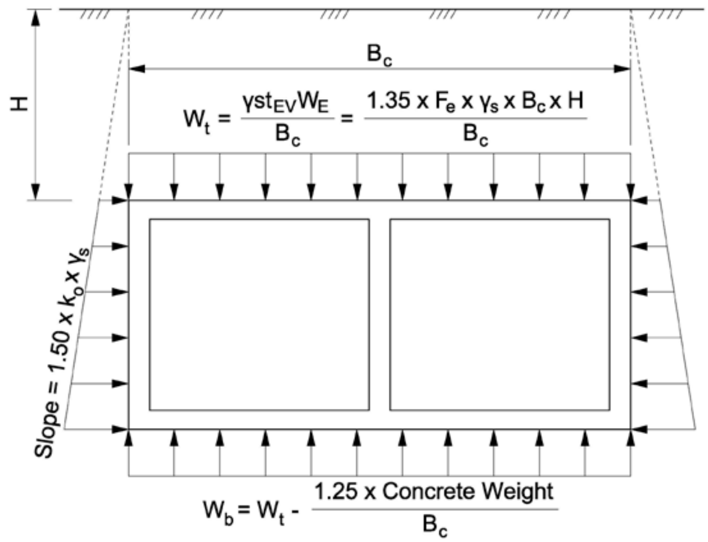


Figure 36.4-1
Factored Vertical and Horizontal Earth Pressures

Where:

W_t	=	Soil pressure on top of box culvert (ksf)
W_b	=	Soil pressure on the bottom of box culvert (ksf)
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)

Figure 36.4-1 shows the factored vertical and horizontal earth load pressures acting on a box culvert. The earth pressure from the dead load of the concrete is distributed equally over the bottom of the box.

36.4.4 Live Load Surcharge (LS)

Per **LRFD [3.11.6.4]**, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the distance from top of pavement to bottom of the box culvert.

Per **LRFD [Table 3.11.6.4-1]**, the following equivalent heights of soil for vehicular loading shall be used. The height to be used in the table shall be taken as the distance from the bottom of the culvert to the roadway surface. Use interpolation for heights other than those listed in the table.

Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Table 36.4-1
Equivalent Height of Soil for Vehicular Loading

Surcharge loads are computed based on a coefficient of lateral earth pressure times the unit weight of soil times the height of surcharge. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil, as discussed in 36.4.3. The uniform distributed load is applied to both exterior walls with the load directed toward the center of the box culvert. The load is designated as, LS, live load surcharge, for application of load factors and limit state combinations. Refer to **LRFD [3.11.6.4]** for additional information regarding live load surcharge.

36.4.5 Water Pressure (WA)

Static water pressure loads are computed when the water height on the outside of the box is greater than zero. The water height is measured from the bottom inside of the box culvert to the water level. The load is designated as, WA, water pressure load, for application of load factors and limit state combinations. Water pressure in culvert barrels is ignored. Refer to **LRFD [3.7.1]** for additional information regarding water pressure.

36.4.6 Live Loads (LL)

Live load consists of the standard AASHTO LRFD trucks and tandem. Per **LRFD [3.6.1.3.3]**, design loads are always axle loads (single wheel loads should not be considered) and the lane load is not used.

Where the depth of fill over the box is less than 2 feet, the wheel loads are distributed per **LRFD [4.6.2.10]**. Where the depth of fill is 2 feet or more, the wheel loads shall be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area **LRFD [3.6.1.2.5]**, increased by the live load distribution factor (LLDF) in **LRFD[Table 3.6.1.2.6a-1]**, using the provisions of **LRFD [3.6.1.2.6b-c]**. Where areas from distributed wheel loads overlap at the top of the culvert, the total load is considered as uniformly distributed over the rectangular area (A_{LL}) defined by the outside limits described in **LRFD [3.6.1.2.6b-c]**.

Per **LRFD [3.6.1.2.6a]**, for single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between inside faces of end walls.

Skew is not considered for design loads.

36.4.6.1 Depth of Fill Less than 2.0 ft.

Where the depth of fill is less than 2.0 ft, follow **LRFD [4.6.2.10]**.

36.4.6.1.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [4.6.2.10.2]**. Use a single lane and the single lane multiple presence factor of 1.2.

Distribution length perpendicular to the span:

$$E = (96 + 1.44(S))$$

Where:

E	=	Equivalent distribution width perpendicular to span (in.)
S	=	Clear span (ft)

The distribution of wheel loads perpendicular to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-2.

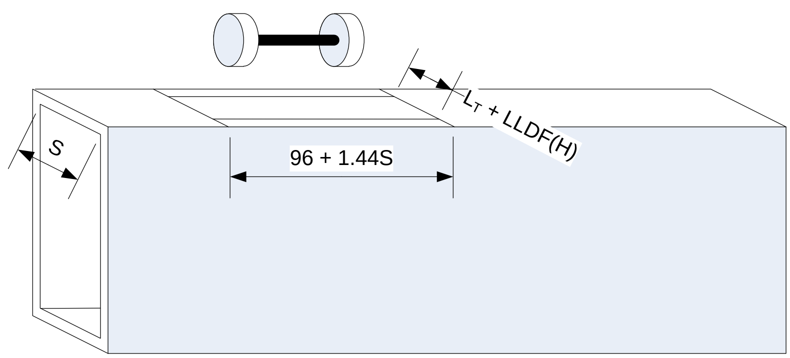


Figure 36.4-2
Distribution of Wheel Loads Perpendicular to Span, Depth of Fill Less than 2.0 feet

Distribution length parallel to the span:

$$E_{span} = (L_T + LLDF\ (H))$$

Where:

E_{span}	=	Equivalent distribution length parallel to span (in.)

L_T = Length of tire contact area parallel to span, as specified in **LRFD [3.6.1.2.5]** (in.)

LLDF = Factor for distribution of live load with depth of fill, 1.15, as specified in **LRFD [Table 3.6.1.2.6a-1]**.

H = Depth of fill from top of culvert to top of pavement (in.)

The distribution of wheel loads parallel to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-3.

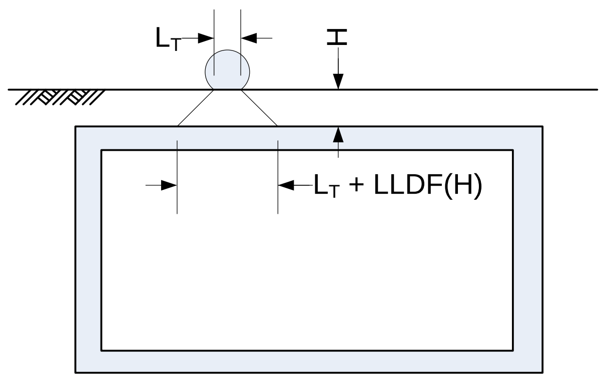


Figure 36.4-3
Distribution of Wheel Loads Parallel to Span, Depth of Fill Less than 2.0 feet

36.4.6.1.2 Case 2 - Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab using the equations specified in **LRFD [4.6.2.1]** for concrete decks with primary strips perpendicular to the direction of traffic per **LRFD[4.6.2.10.3]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

For a cast-in-place box culvert, the width of the primary strip, in inches is:

+M: 26.0 + (6.6)(S)

-M: 48.0 + (3.0)(S)

as stated in **LRFD [Table 4.6.2.1.3-1]**

Where:

S = Spacing of supporting components (ft)

+M = Positive moment

-M	=	Negative moment

36.4.6.2 Depth of Fill Greater than or Equal to 2.0 ft.

Where the depth of fill is 2.0 ft or greater, follow **LRFD [3.6.1.2.6b-c]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

36.4.6.2.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [3.6.1.2.6b]**.

For live load distribution transverse to span, the wheel/axle load interaction depth, H_{int-t}, shall be:

$$H_{int-t} = \frac{S_w - W_t/12 - 0.06D/12}{LLDF} \qquad \text{(ft)}$$

where $H < H_{int-t}$ (no lateral interaction); then $W_w = W_t/12 + LLDF \cdot (H) + 0.06 \cdot (D/12)$

where $H \geq H_{int-t}$ (lateral interaction); then $W_w = W_t/12 + S_w + LLDF \cdot (H) + 0.06 \cdot (D/12)$

For live load distribution parallel to span, the wheel/axle load interaction depth H_{int-p} shall be:

$$H_{int-p} = \frac{S_a - \ell_t/12}{LLDF} \qquad \text{(ft)}$$

where $H < H_{int-p}$ (no longit. interaction); then $\ell_w = \ell_t/12 + LLDF \cdot (H)$

where $H \geq H_{int-p}$ (longit. interaction); then $\ell_w = \ell_t/12 + S_a + LLDF \cdot (H)$

Where:

D	=	Clear span of the culvert (in)
H	=	Depth of fill from top of culvert to top of pavement (in)
H_{int-t}	=	Wheel interaction depth transverse to span (ft)
H_{int-p}	=	Axle interaction depth parallel to span (ft)
LLDF	=	Live load distribution factor per **LRFD [Table 3.6.1.2.6a-1]**; (1.15)
W_t	=	Width of tire contact area, per **LRFD [3.6.1.2.5]**; (20 in)
ℓ_t	=	Length of tire contact area, per **LRFD [3.6.1.2.5]**; (10 in)
S_w	=	Wheel spacing; (6.0 ft)

S_a = Axle spacing (ft)

W_w = Live load patch width at depth H (ft)

ℓ_w = Live load patch length at depth H (ft)

$A_{LL} = \ell_w \cdot W_w$

Where:

A_{LL} = Rectangular area at depth H (ft^2)

The live load vertical crown pressure shall be:

$$P_L = \frac{P(1 + IM/100)(m)}{A_{LL}}$$

Where:

IM = Dynamic load allowance (%); (see 36.4.8)

m = Multiple presence factor per **LRFD [3.6.1.1.2]**

P = Live load applied at surface on all interacting wheels (kip)

P_L = Live load vertical crown pressure (ksf)

The longitudinal and transverse distribution widths for depths of fill greater than or equal to 2.0 feet are illustrated in Figure 36.4-4.

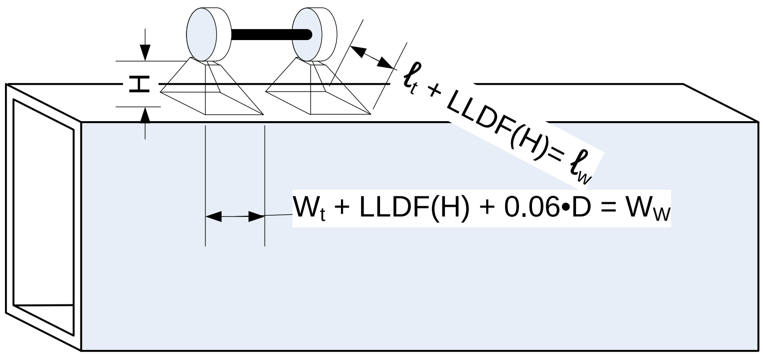


Figure 36.4-4
Distribution of Wheel Loads, Depth of Fill $\geq$ 2.0 feet (no lateral interaction)

36.4.6.2.2 Case 2 – Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab as described in **LRFD [3.6.1.2.6c]**.

36.4.7 Live Load Soil Pressures

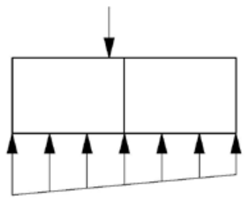


Figure 36.4-5
Vertical Soil Pressure under Culvert

The soil pressure on the bottom of the box is determined by moving the live load across the box. Find the location where the live load causes the maximum effects on the top slab of the box. At that location, determine the soil pressure diagram that will keep the system in equilibrium. Use the effects of this soil pressure in the bottom slab analysis.

36.4.8 Dynamic Load Allowance

Dynamic load allowance decreases as the depth of fill increases. **LRFD [3.6.2.2]** states that the impact on buried components shall be calculated as:

$$IM = 33(1.0 - 0.125(D_E)) \geq 0\%$$

Where:

D_E = Minimum depth of earth cover above the structure (ft)

36.4.9 Location for Maximum Moment

Create influence lines and use notional loading to determine the location for maximum moment. In this analysis, include cases for variable axle spacing and reverse axle order for unsymmetrical loading conditions.

For notional vehicles, only the portion of the loading that contributes to the effect being maximized is included. This is illustrated in Figure 36.4-6.

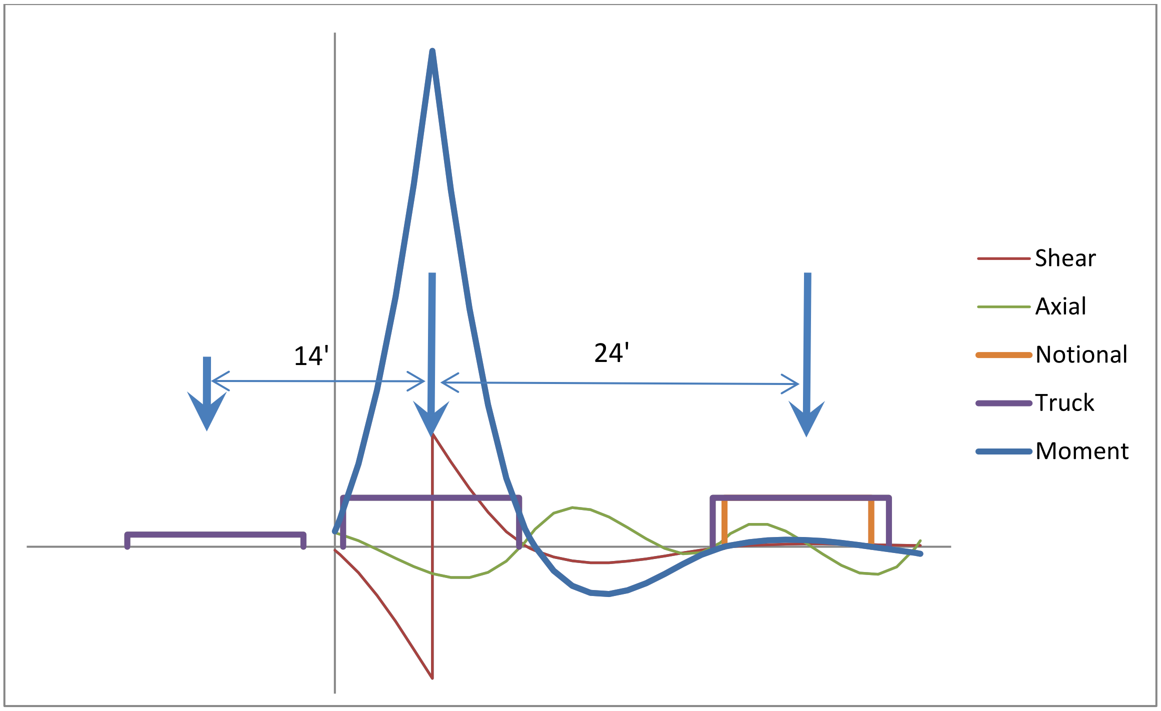


Figure 36.4-6
Application of Notional Loading using Influence Lines

The maximum positive moment results when the middle axial load is centered at the first positive peak while the variable rear axial spacing is 24 feet. Only the portion of the rear axial load in the positive region of the moment influence line is considered. The middle axial load and the portion of the rear axial in the positive region of the moment influence line are loaded on the shear and axial influence lines to compute the corresponding effects. Both positive and negative portions of the shear and axial influence lines are used when computing the corresponding effects. This process is repeated for maximizing the negative moment, shear and axial effects and computing the corresponding effects.

36.5 Design Information

Sidesway of the box is not considered because of the lateral support of the soil.

The centerline of the walls and top and bottom slabs are used for computing section properties and dimensions for analysis.

WisDOT Policy Item:

For skews 20 degrees or less, place the reinforcing steel along the skew. For skews over 20 degrees, place the reinforcing steel perpendicular to the centerline of box.

Culverts are analyzed as if the reinforcing steel is perpendicular to the centerline of box for all skew angles.

The minimum thickness of the top and bottom slab is 6½ inches. For pedestrian underpasses and slabs with fills less than 2 feet, the minimum thickness of the top slab should be 1 foot. Minimum wall thickness is based on the inside opening of the box (height) and the height of the apron wall above the floor. Use the following table to select the minimum wall thickness that meets or exceeds the three criteria in the table.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

All slab thicknesses are rounded to the next largest ½ inch.

Top and bottom slab thicknesses are determined by shear and moment requirements. Slab thickness shall be adequate to carry the factored shear without shear reinforcement.

All bar steel is detailed as being 2 inches clear with the following exceptions:

- The bottom steel in the bottom slab is detailed with 3 inches clear
- The top steel in the top slab of a box culvert with no fill is detailed with 2½ inches clear

A haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Only 45º haunches shall be used. Minimum haunch depth and length is 6 inches. Haunch dimensions are increased in 3 inch increments.

The slab thickness required is determined by moment or shear, whichever governs.

The shear in the top and bottom slabs is assumed to occur at a distance "d" from the face of the walls. The value for "d" equals the distance from the centroid of the reinforcing steel to the face of the concrete in compression. When a haunch is used, shear must also be checked at the end of the haunch.

For multi-cell culverts make interior and exterior walls of equal thickness.

Culverts shall be designed for the range of fill between the shoulders of the roadway. To accommodate future widening of the roadway, reduced sections may not be used on the ends of the culvert where there is less fill. Exceptions may be made with the approval of the Bureau of Structures where the culvert has high fills and a reduced section could be used for at least two panel pours per end of culvert. Culvert extensions shall be designed for the same range of fills as the original culvert. The extension design shall not have lower capacity than the original culvert. Maximum length of panel pour is 40 feet.

Barrel lengths are based on the roadway sections and wing lengths are based on a minimum 2 1/2:1 slope of fill from the top of box to apron. Consideration shall be given to match the typical roadway cross slope.

Dimensions on drawings are given to the nearest ¼ inch only.

36.6 Detailing of Reinforcing Steel

To calculate the required bar steel area and cutoff points a maximum positive and negative moment envelope is computed. It is assumed that the required bar lengths in the top slab are longer than those in the bottom slab. Therefore, cutoff points are computed for the top slab and are also used in the bottom slab.

36.6.1 Bar Cutoffs

Per **LRFD [5.11.1.2.1]**, all flexural reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

- The effective depth of the member
- 15 times the nominal diameter of the bar
- 1/20 of the clear span

Continuing reinforcement shall extend not less than the development length, ℓ_d (**LRFD [5.11.2]**) beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

Per **LRFD [5.11.1.2.2]**, at least one-third of the positive moment reinforcement in simple span members and one-fourth of the positive moment reinforcement in continuous span members shall extend along the same face of the member beyond the centerline of the support. In beams, such extension shall not be less than 6.0 in.

Per **LRFD [5.11.1.2.3]**, at least one-third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than:

- The effective depth of the member
- 12 times the nominal diameter of the bar
- 0.0625 times the clear span

36.6.2 Corner Steel

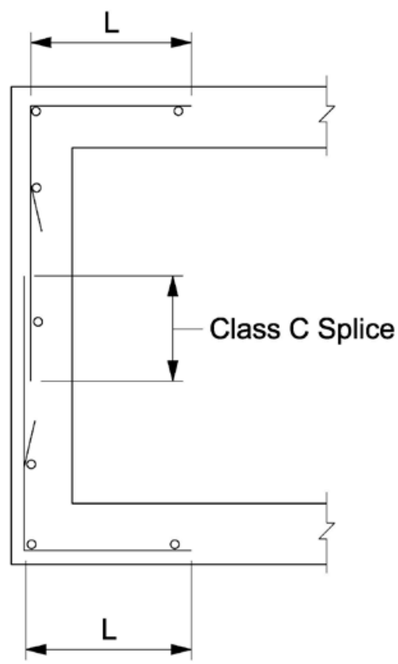


Figure 36.6-1
Layout of Corner Steel

The area of steel required is the maximum computed from using the top and bottom corner moments and the thickness of the slab or wall, whichever controls. Identical bars are used in the top and bottom corners. Identical length bars are used in the left and right corners if the bar lengths are within 2 feet of one another. Top and bottom negative steel is cut in the walls and detailed in two alternating lengths when a savings of over 2 feet in a single bar length can be obtained. Corner steel is always lapped at the center of the wall. If two bar lengths are used, only alternate bars are lapped.

Distance "L" is computed from the maximum negative moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1.

36.6.3 Positive Moment Slab Steel

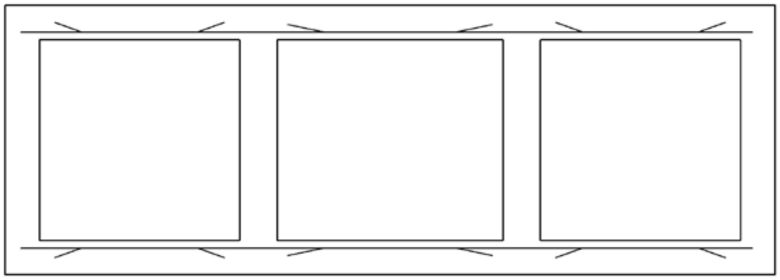


Figure 36.6-2
Layout of Positive Moment Steel

The area of steel required is determined by the maximum positive moments in each span. Top and bottom slab reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

When two alternating bar lengths are detailed in multi-cell culverts, run every other positive bar across the entire width of box. If this requires a length longer than 40 feet, lap them over an interior wall. For 2 or more cells, if the distance between positive bars of adjacent cells is 1 foot or less, make the bar continuous.

The cutoff points of alternate bars are determined from the maximum positive moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1. These same points are used in the bottom slab. Identical bar lengths are used over multiple cells if bars are within 2 feet of one another.

36.6.4 Negative Moment Slab Steel over Interior Walls

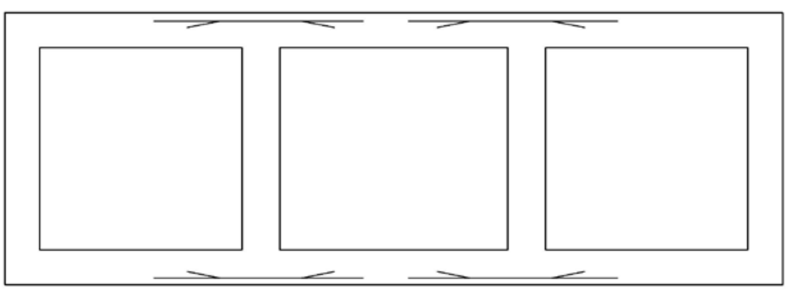


Figure 36.6-3
Layout of Negative Moment Steel

If no haunch is present, the area of steel required is determined by using the moment and effective depth at the face of the interior wall. If the slab is haunched, the negative reinforcement is determined per **LRFD [12.11.4.2],** which states that the negative moment is determined at the intersection of the haunch and uniform depth member. Top and bottom slab reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

Cutoff points are determined from the maximum negative moment envelope of the top slab and shall include the extension lengths discussed in 36.6.1. The same bar lengths are then used in the bottom slab. Identical bar lengths are used over multiple interior walls if bars are within 2 feet of one another. The minimum length of any bar is 2 times the development length. For culverts of 3 or more cells, if the clear distance between negative bars of adjacent spans is 1 foot or less, make the bar continuous across the interior spans.

When there is no fill over the top slab, run the negative moment reinforcing steel across the entire width of the culvert. Refer to 36.6.8 for temperature and shrinkage requirements.

36.6.5 Exterior Wall Positive Moment Steel

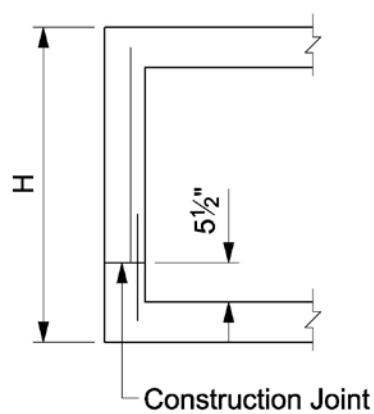


Figure 36.6-4
Layout of Exterior Wall Steel

The area of steel is determined by the maximum positive moment in the wall. A minimum of #4 bars at 18 inches is supplied. The wall bar is extended to 2 inch top clear and the dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed.

36.6.6 Interior Wall Moment Steel

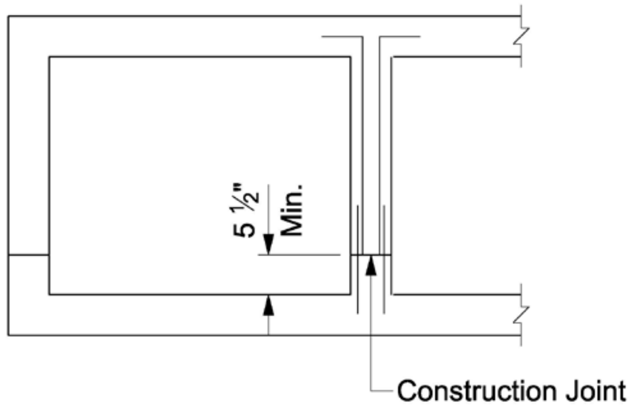


Figure 36.6-5
Layout of Interior Wall Steel

The area of steel is determined from the maximum moment at the top of the wall and the effective wall thickness. A minimum of #4 bars at 18 inches is supplied. Identical steel is provided at both faces of the wall. A 1 foot, 90 degree bend, is provided in the top slab with the horizontal portion being just below the negative moment steel. The dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed. When a haunch is provided, the construction joint is placed a distance above the bottom slab equal to the haunch depth plus 2 inches.

36.6.7 Distribution Reinforcement

Per **LRFD [5.14.4.1]**, transverse distribution reinforcement is not required for culverts where the depth of fill exceeds 2.0 feet.

Per **LRFD [12.11.2.1]**, provide distribution reinforcement for culverts with less than or equal to 2 feet of fill in accordance with **LRFD [9.7.3.2]**, which states that reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows (for primary reinforcement parallel to traffic):

$$\text{Percentage} = \frac{100}{\sqrt{S}} \leq 50\%$$

Where:

S = Effective span length (ft) (for slabs monolithic with walls, this distance is taken as the face-to-face distance per **LRFD [9.7.2.3]**)

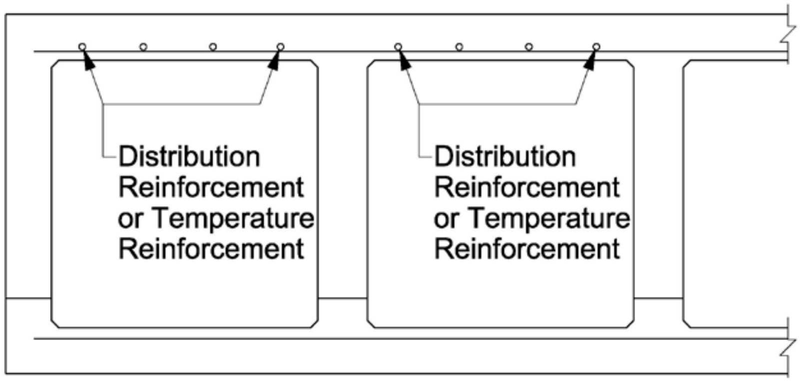


Figure 36.6-6
Layout of Distribution Steel

36.6.8 Shrinkage and Temperature Reinforcement

Shrinkage and temperature reinforcement is required on all inside culvert faces, negative moment regions in top slabs, and on both wingwall faces in each direction that does not already have strength or distribution reinforcement. Shrinkage and temperature reinforcement is not required on the outside (soil) face for culvert barrels. This includes exterior walls, the bottom of the bottom slab, and in some cases the top face of the top slab in the positive moment region. Per **LRFD [12.11.4.3.1]**, provide shrinkage and temperature reinforcement near the inside surfaces of walls and slabs in accordance with **LRFD [5.10.8]**, which states that the area of shrinkage and temperature steel per foot on each face and in each direction shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$

$$0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	Area of reinforcement in each direction and each face (in^2/ft)
b	=	Least width of component section (in.)
h	=	Least thickness of component section (in.)
f_y	=	Specified yield strength of reinforcing bars ≤ 75 (ksi)

Where the least dimension varies along the length of the component, multiple sections should be examined to represent the average condition at each section.

Shrinkage and temperature reinforcement shall use a minimum of #4 bars at 18 inch centers in both directions.

36.7 Box Culvert Aprons

Five types of box culvert aprons are used. They are referred to as Type A, B, C, D and E. The angle that the wings make with the direction of stream flow is the main difference between the five types. The allowable headwater and other hydraulic requirements are what usually determine the type of apron required. Physical characteristics at the site may also dictate a certain type. For hydraulic design of different apron types see Chapter 8.

36.7.1 Type A

Type A, because of its poor hydraulic properties, is generally not used except for cattle or pedestrian underpasses.

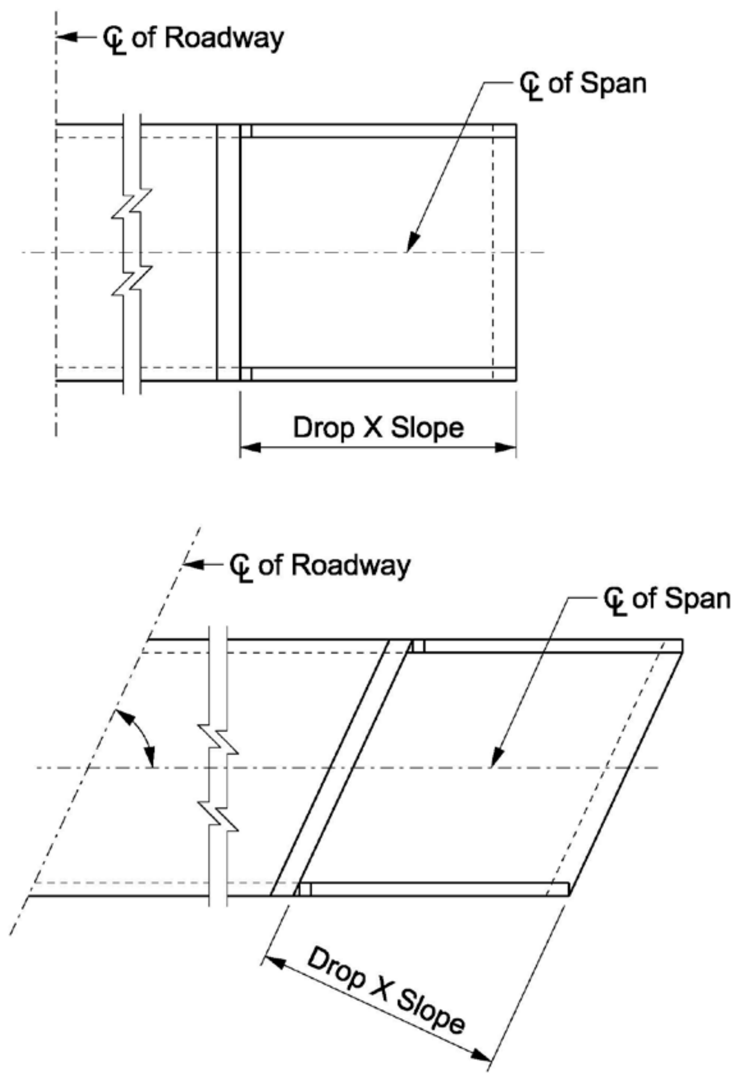
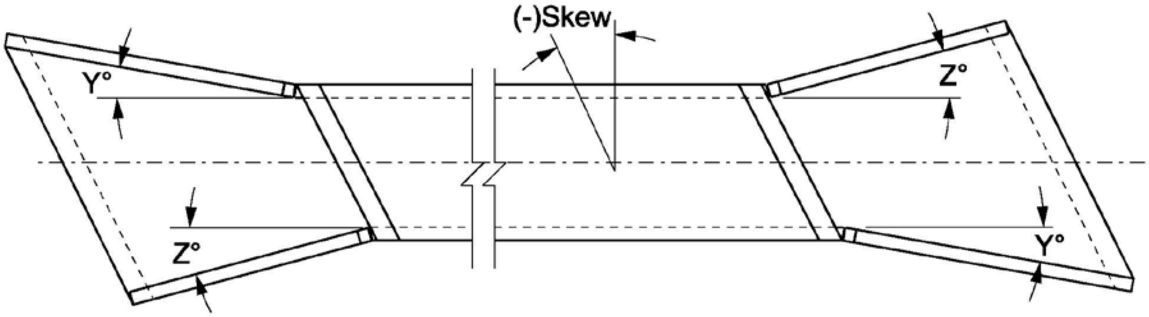


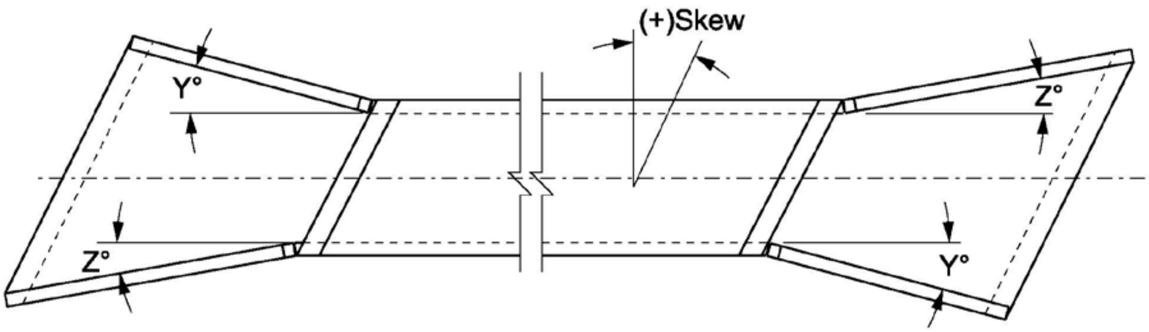
Figure 36.7-1
Plan View of Type A

36.7.2 Type B, C, D

Type B is used for outlets. Type C & D are of equal efficiency but Type C is used most frequently. Type D is used for inlets when the water is entering the culvert at a very abrupt angle. See Figure 36.7-2 for Wing Type B, C and D for guidance on wing angles for culvert skews.



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	25°	30°	40°	45°
15.0°	22.5°	10°	15°	25°	30°	35°	45°
22.5°	37.5°	10°	15°	20°	30°	30°	45°
37.5°	45.0°	10°	15°	15°	30°	25°	45°
45.0°	52.5°	5°	15°	15°	30°	20°	45°
52.5°	67.5°	5°	15°	10°	30°	15°	45°
67.5°	75.0°	5°	15°	5°	30°	10°	45°
75.0°	82.5°	0°	15°	5°	30°	5°	45°
82.5°	90.0°	0°	15°	0°	30°	0°	45°



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	30°	25°	45°	40°
15.0°	22.5°	15°	10°	30°	25°	45°	35°
22.5°	37.5°	15°	10°	30°	20°	45°	30°
37.5°	45.0°	15°	10°	30°	15°	45°	25°
45.0°	52.5°	15°	5°	30°	15°	45°	20°
52.5°	67.5°	15°	5°	30°	10°	45°	15°
67.5°	75.0°	15°	5°	30°	5°	45°	10°
75.0°	82.5°	15°	0°	30°	5°	45°	5°
82.5°	90.0°	15°	0°	30°	0°	45°	0°

Figure 36.7-2
Wing Type B, C, D (Angles vs. Skew)

36.7.3 Type E

Type E is used primarily in urban areas where a sidewalk runs over the culvert and it is necessary to have a parapet and railing along the sidewalk. For Type E the wingwalls run parallel to the roadway just like the abutment wingwalls of most bridges. It is also used where Right of Way (R/W) is a problem and the aprons would extend beyond the R/W for other types. Wingwall lengths for Type E wings are based on a minimum channel side slope of 1.5 to 1.

36.7.4 Wingwall Design

Culvert wingwalls are designed using a 1 foot surcharge height, a unit weight of backfill of 0.120 kcf and a coefficient of lateral earth pressure of 0.5, as discussed in 36.4.3. When the wingwalls are parallel to the direction of traffic and where vehicular loads are within ½ the wall height from the back face of the wall, design using a surcharge height representing vehicular load per **LRFD [Table 3.11.6.4-2]**. Load and Resistance Factor Design is used, and the load factor for lateral earth pressure of γ_{EH} = 1.69 is used, based on past design experience. The lateral earth pressure was conservatively selected to keep wingwall deflection and cracking to acceptable levels. Many wingwalls that were designed for lower horizontal pressures have experienced excessive deflections and cracking at the footing. This may expose the bar steel to the water that flows through the culvert and if the water is of a corrosive nature, corrosion of the bar steel will occur. This phenomena has led to complete failure of some wingwalls throughout the State.

For wing heights of 7 feet or less determine the area of steel required by using the maximum wall height and use the same bar size and spacing along the entire wingwall length. The minimum amount of steel used is #4 bars at 12 inch spacing. Wingwall thickness is made equal to the barrel wall thickness.

For wing heights over 7 feet the wall length is divided into two or more segments and the area of steel is determined by using the maximum height of each segment. Use the same bar size and spacing in each segment.

Wingwalls must satisfy Strength I Limit State for flexure and shear, and Service I Limit State for crack control, minimum reinforcement, and reinforcement spacing. Adequate shrinkage and temperature reinforcement shall be provided.

36.8 Box Culvert Camber

Camber of culverts is a design compensation for anticipated settlement of foundation soil beneath the culvert. Responsibility for the recommendation and calculation of camber belongs to the Regional Soils Engineer. Severe settlement problems with accompanying large camber are to be checked with the Geotechnical Section.

Both total and differential settlement need to be considered to determine the amount of box camber required to avoid adverse profile sag and undesirable separation at culvert joints per **LRFD [12.6.2.2]**. If the estimated settlement is excessive, contingency measures will need to be considered, such as preloading with embankment surcharge, undercutting and subgrade stabilization. To evaluate differential settlement, it will be necessary to calculate settlement at more than one point along the length of the box culvert.

36.8.1 Computation of Settlement

Settlement should be evaluated at the Service Limit state in accordance with **LRFD [12.6.2.2]** and **LRFD [10.6.2]**, and consider instantaneous elastic consolidation and secondary components. Elastic settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. Consolidation settlement is the gradual compression of the soil skeleton when excess pore pressure is forced out of the voids in the soil. Secondary settlement, or creep, occurs as a result of plastic deformation of the soil skeleton under constant effective stress. Secondary settlement is typically not significant for box culvert design, except where there is an increase in effective stress within organic soil, such as peat. If secondary settlement is a concern, it should be estimated in accordance with **LRFD [10.6.2.4]**.

Total settlement, including elastic, consolidation and secondary components may be taken in accordance with **LRFD [10.6.2.4.1]** as:

$$S_t = S_e + S_c + S_s$$

Where:

S_t	=	Total settlement (ft)
S_e	=	Elastic settlement (ft)
S_c	=	Primary consolidation settlement (ft)
S_s	=	Secondary settlement (ft)

To compute settlement, the subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about 3 times the box width. The maximum layer thickness should be 10 feet.

Primary consolidation settlement for normally-consolidated soil is computed using the following equation in accordance with **LRFD [10.6.2.4.3]**:

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right]$$

Where:

S_c	=	Primary consolidation settlement (ft)
H_c	=	Initial height of compressible soil layer (ft)
e_o	=	Void ratio at initial vertical effective stress
C_c	=	Compression index which is a measure of the compressibility of a soil. It is the slope of the straight-line part of the e-log p curve from a conventional consolidation (oedometer) test.
σ'_f	=	Final vertical effective stress at midpoint of soil layer under consideration (ksf)
σ'_o	=	Initial vertical effective stress at midpoint of soil layer under consideration (ksf)

If the soil is over-consolidated, reference is made to **LRFD [10.6.2.4.3]** to estimate consolidation settlement.

Further description for the above equations and consolidation test can be found in most textbooks on soil mechanics.

For preliminary investigations C_c can be determined from the following approximate formula, found in most soil mechanics textbooks:

Non organic soils: $C_c = 0.007\ (LL-10)$

Where:

LL	=	Liquid limit expressed as whole number.

If the in-place moisture content approaches the plastic limit the computed C_c is decreased by 75%. If the in-place moisture content is near the liquid limit use the computed value. If the in-place moisture content is twice the liquid limit the computed C_c is increased by 75%. For intermediate moisture contents the percent change to the computed C_c is determined from a straight line interpolation between the corrections mentioned above.

If settlements computed by using the approximate value of C_c exceed 1.5 feet, a consolidation test is performed. As in-place moisture content approaches twice the liquid limit, settlement is caused by a local shear failure and the consolidation equation is no longer applicable.

The consolidation equation is applied to only compressible silts and clays. Sands are of a lower compressibility and no culvert camber is required until the fill exceeds 25 feet. When the fill exceeds 25 feet for sand, a camber of 0.01 feet per foot of fill is used.

36.8.2 Configuration of Camber

The following guides are to be followed when detailing camber.

- It is unnecessary to provide gradual camber. "Brokenback" camber is closer to the actual settlement which occurs.
- Settlement is almost constant from shoulder point to shoulder point. It then reduces to the ends of the culvert at the edge of the fill.
- The ends of the culvert tend to come up if side slopes are steeper than 2½ to 1. With 2 to 1 side slopes camber is increased 10% to compensate for this rise.

36.8.3 Numerical Example of Settlement Computation

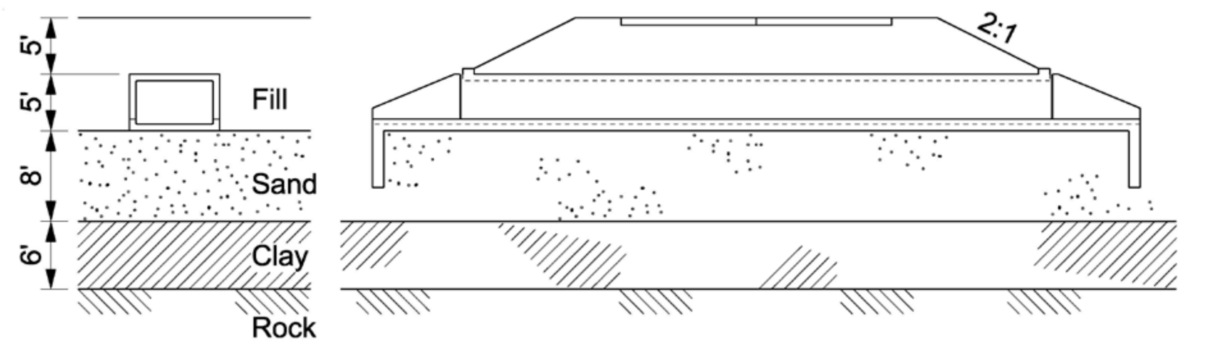


Figure 36.8-1
Soil Strata under Culvert

A box culvert rests on original ground consisting of 8 feet of sand and 6 feet of clay over bedrock. Estimate the settlement of the culvert if 10 feet of fill is placed on the original ground after the culvert is constructed. The in-place moisture content and liquid limit equal 40%. The initial void ratio equals 0.98. The unit weight of the clay is 105 pcf and that of the fill and sand is 110 pcf. There is no water table.

σ'_o = (8 ft)(110 pcf) + (3 ft)(105 pcf) = 1195 psf

$\sigma'_f = \sigma'_o$ + (10 ft)(110 pcf) = 1195 psf + 1100 psf = 2295 psf

C_c = 0.007 (40-10) = 0.21 (approximate value)

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right] = \frac{6\text{ft}}{1+0.98} 0.21 * \log_{10}\left[\frac{2295\text{psf}}{1195\text{psf}}\right] = 0.18\text{ft}$$

36.9 Box Culvert Structural Excavation and Structure Backfill

All excavations for culverts and aprons, unless on bedrock or fill, are to include a 6 inch minimum undercut and backfilled with structural backfill, as described in the specification. This undercut is for construction purposes and provides a solid base for placing reinforcement and pouring the bottom slab. For fill sections, it is assumed that placed fills provide a solid base and structural backfill is not needed. For cut sections, deeper under cuts may be warranted to mitigate differential settlement.

All volume excavated and not occupied by the new structure should be backfilled with structure backfill for the full length of the box culvert, including the apron.

See Standard Detail 9.01 – Structure Backfill Limits and Notes – for typical pay limits and plan notes.

36.10 Box Culvert Headers

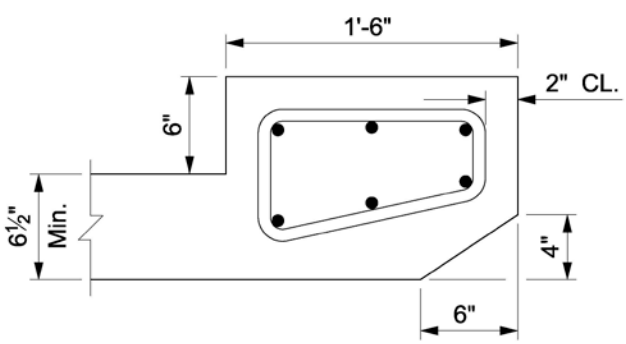
For skews of 20 degrees and less the main reinforcing steel is parallel to the end of the barrel. A header is not required for structural purposes but is used to prevent the fill material from spilling into the apron. A 12 inch wide by 6 inch high (above the top of top slab) header with nominal steel is therefore used for skews of 20 degrees and less on the top slab. No header is used on the bottom slab.

For skews over 20 degrees the main reinforcing is not parallel to the end of the barrel. The positive reinforcing steel terminates in the header and thus the header must support, in addition to its own dead load, an additional load from the dead load of the slab and fill above it. A portion of the live load may also have to be supported by the header.

The calculation of the actual load that a header must support becomes a highly indeterminate problem. For this reason a rational approach is used to determine the amount of reinforcement required in the headers. The design moment capacity of the header must be equal to or greater than 1.25 times the header dead load moment (based on simple span) plus 1.75 times a live load moment from a 16 kip load assuming 0.5 fixity at ends.

To prevent a traffic hazard, culvert headers are designed not to protrude above the ground line. For this reason the height of the header above the top of the top slab is typically selected to be 6 inches. The width of the header is standardized at 18 inches.

The header in the following figure gives the design moment capacities listed using d = 8.5 inches.



Bar Size	Moment Capacity ft.-kip
#6	46.1
#7	60.7
#8	76.5
#9	92.1
#10	109.1

Figure 36.10-1
Header Details (Skews > 20°)

The following size bars are recommended for the listed header lengths where "Header Length" equals the distance between C/L of walls in one cell measured along the skew.

Header Length	Bar Size [1]
To 11’	#7
Over 11’ to 14’	#8
Over 14’ to 17’	#9
Over 17’ to 20’	#10

Table 36.10-1
Header Reinforcement

[1] Use the bar size listed in each header and place 3 bars on the top and 3 bars on the bottom. Use a header on both the top and bottom slab. See the Standard *Box Culvert Details* in Chapter 36.

Where headers greater than 6 inches in height are used to retain roadway fill, the top slab shall be designed to handle the bending moment transmitted from the header. Additional reinforcement may be required.

Where barriers are placed on top of the culvert header, the barrier, header, and top slab shall be designed for vehicular impact forces.

36.11 Plan Detailing Issues

36.11.1 Weep Holes

Investigate the need for weep holes for culverts in cohesive soils. These holes are to relieve the hydrostatic pressure on the sides of the culverts. Where used, place the weep holes 1 foot above normal water elevation but a minimum of 1 foot above the lower sidewall construction joint. Do not place weep holes closer than 1 foot from the bottom of the top slab.

36.11.2 Cutoff Walls

Where dewatering the cutoff wall in sandy terrain is a problem, the concrete may be poured in the water. Place a note on the plans allowing concrete for the cutoff wall to be placed in the water.

36.11.3 Nameplate

Designate a location on the wingwall for placement of the nameplate. Locate nameplate on the first right wing traveling in the Cardinal direction (North/East).

36.11.4 Plans Policy

If a cast-in-place reinforced concrete box culvert is used, full plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

36.11.5 Rubberized Membrane Waterproofing

When required by the Standard Details, place the bid item "Rubberized Membrane Waterproofing" on the final plans. The quantity is given square yards.

36.12 Precast Four-Sided Box Culverts

In general, structural contractors prefer cast-in-place culverts while grading contractors prefer precast culverts. Precast culverts have been more expensive than cast-in-place culverts in the past, but allow for reduced construction time. Box culverts that are 4 feet wide by 6 feet high or less are considered roadway culverts. All other culverts require a B or C number along with the appropriate plans. All culverts requiring a number should be processed through the Bureau of Structures.

When a precast culvert is selected as the best structure type for a particular project during the design study phase, preliminary plans and complete detailed final plans are required to be sent to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

Sometimes a complete set of plans is created for a cast-in-place culvert and a precast culvert is stated to be an acceptable alternate. If the contractor selects the precast alternate, the contractor is to submit shop drawings, sealed by a professional engineer, to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

36.13 Three-Sided Structures

Three-sided box culvert structures are divided into two categories: cast-in-place three sided structures and precast three-sided structures. These structures shall follow the criteria outlined below.

36.13.1 Cast-In-Place Three-Sided Structures

To be developed

36.13.2 Precast Three-Sided Structures

Three-sided precast concrete structures offer a cost effective, convenient solution for a variety of bridge needs. The selection of whether a structure over a waterway should be a culvert, a three-sided precast concrete structure or a bridge is heavily influenced by the hydraulic opening. As the hydraulic opening becomes larger, the selection process for structure type progresses from culvert to three-sided precast concrete structure to bridge. Cost, future maintenance, profile grade, staging, skew, soil conditions and alignment are also important variables which should be considered. Culverts generally have low future maintenance; however, culverts should not be considered for waterways with a history or potential of debris to avoid channel cleanout maintenance. In these cases a three-sided precast concrete structure may be more appropriate. Three-sided precast concrete structures have the advantage of larger single and multiple openings, ease of construction, and low future maintenance costs.

A precast-concrete box culvert may be recommended by the Hydraulics Team. The side slope at the end or outcrop of a box culvert should be protected with guardrail or be located beyond the clear zone.

The hydraulic recommendations will include the Q_{100} elevation, the assumed flowline elevation, the required span, and the required waterway opening for all structure selections. The designer will determine the rise of the structure for all structure sections.

A cost comparison is required to justify a three-sided precast concrete structure compared to other bridge/culvert alternatives.

To facilitate the initiation of this type of project, the BOS is available to assist the Owners and Consultants in working out problems which may arise during plan development.

Some of the advantages of precast three-sided structures are listed below:

- Speed of Installation: Speed of installation is more dependent on excavation than product handling and placement. Precast concrete products arrive at the jobsite ready to install. Raw materials such as reinforcing steel and concrete do not need to be ordered, and no time is required on site to set up forms, place concrete, and wait for the concrete to cure. Precast concrete can be easily installed on-demand and immediately backfilled.

- Environmentally Friendly: Precast concrete is ready to be installed right off the delivery truck, which means less storage space needed for scaffolding and rebar. There is less noise pollution from ready-mix trucks continually pulling up on site and less waste as a result of using precast (i.e. no leftover steel, no pieces of scaffolding and no waste concrete piles). The natural bottom on a three-sided structure is advantageous to meet fish passage and DNR requirements.

- Quality Control: Because precast concrete products are produced in a quality-controlled environment with proper curing conditions, these products exhibit higher quality and uniformity over cast-in-place structures.

- Reduced Weather Dependency: Weather does not delay production of precast concrete as it can with cast-in-place concrete. Additionally, weather conditions at the jobsite do not significantly affect the schedule because the "window" of time required for installation is small compared to other construction methods, such as cast-in-place concrete.

- Maintenance: Single span precast three-sided structures are less susceptible to clogging from debris and sediment than multiple barrel culvers with equivalent hydraulic openings.

36.13.2.1 Precast Three-Sided Structure Span Lengths

WisDOT BOS allows and provides standard details for the following precast three-sided structure span lengths:

14’-0, 20’-0, 24’-0, 28’-0, 36’-0, 42’-0

Dimensions, rises, and additional guidance for each span length are provided in the standard details.

36.13.2.2 Segment Configuration and Skew

It is not necessary for the designer to determine the exact number and length of segments. The final structure length and segment configuration will be determined by the fabricator and may deviate from that implied by the plans.

A zero degree skew is preferable but skews may be accommodated in a variety of ways. Skew should be rounded to the nearer most-practical 5 deg., although the nearer 1 deg. is permissible where necessary. The range of skew is dependent on the design span and the fabrication limitations. Some systems are capable of fabricating a skewed segment up to a maximum of 45 degrees. Other systems accommodate skew by fabricating a special trapezoidal segment. If adequate right-of-way is available, skewed projects may be built with all right angle segments provided the angle of the wingwalls are adjusted accordingly. The designer shall consider the layout of the traffic lanes on staged construction projects when determining whether a particular three-sided precast concrete structure system is suitable.

Square segments are more economical if the structure is skewed. Laying out the structure with square segments will result in the greatest right-of-way requirement and thus allow ample space for potential redesign by the contractor, if necessary, to another segment configuration.

For a structure with a skew less than or equal to 15 deg., structure segments may be laid out square or skewed. Skewed segments are preferred for short structures (approximately less than 80 feet in length). Square segments are preferred for longer structures. However, skewed segments have a greater structural span. A structure with a skew of greater than 15 deg. requires additional analysis per the AASHTO LRFD Bridge Design Specifications. Skewed segments and the analysis both contribute to higher structure cost.

For a structure with a skew greater than 15 deg, structure segments should be laid out square. The preferred layout scheme for an arch-topped structure with a skew of greater than 15 deg should assume square segments with a sloping top of headwall to yield the shortest possible wingwalls. Where an arch-topped structure is laid out with skewed ends (headwalls parallel to the roadway), the skew will be developed within the end segments by varying the lengths of the legs as measured along the centerline of the structure. The maximum attainable skew is controlled by the difference between the full-segment leg length as recommended by the arch-topped-structure fabricator and a minimum leg length of 2 feet.

36.13.2.3 Minimum Fill Height

Minimum fill over a precast three-sided structure shall provide sufficient fill depth to allow adequate embedment for any required beam guard plus 6”. Refer to Standard 36.10 for further information.

Barriers mounted directly to the precast units are not allowed, as this connection has not been crash tested.

36.13.2.4 Rise

The maximum rises of individual segments are shown on the standard details. This limit is based on the fabrication forms and transportation. The maximum rise of the segment may also be limited by the combination of the skew involved because this affects transportation on the truck. Certain rise and skew combinations may still be possible but special permits may be required for transportation. The overall rise of the three-sided structure should not be a limitation when satisfying the opening requirements of the structure because the footing is permitted to extend above the ground to meet the bottom of the three-sided segment.

36.13.2.5 Deflections

Per **LRFD [2.5.2.6.2]**, the deflection limits for precast reinforced concrete three-sided structures shall be considered mandatory.

36.13.3 Plans Policy

If a precast or cast-in-place three-sided culvert is used, full design calculations and plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

The designer should use the span and rise for the structure selection shown on the plans as a reference for the information required on the title sheet. The structure type to be shown on the Title, Layout and General Plan sheets should be Precast Reinforced Concrete Three-Sided Structure.

The assumed elevations of the top of the footing and the base of the structure leg should be shown. For preliminary structure layout purposes, a 2-foot footing thickness should be assumed with the base of the structure leg seated 2 inches below the top-of-footing elevation. With the bottom of the footing placed at the minimum standard depth of 4 feet below the flow line elevation, the base of the structure leg should therefore be shown as 2’-2” below the flow line. An exception to the 4-foot depth will occur where the anticipated footing thickness is known to exceed 2 feet, where the footing must extend to rock, or where poor soil conditions and scour concerns dictate that the footing should be deeper.

The structure length and skew angle, and the skew, length and height of wingwalls should be shown. For a skewed structure, the wingwall geometrics should be determined for each wing. The sideslope used to determine the wing length should be shown on the plans.

If the height of the structure legs exceeds 10 feet, pedestals should be shown in the structure elevation view.

The following plan requirements shall be followed:

1. Preliminary plans are required for all projects utilizing a three-sided precast concrete structure.

2. Preliminary and Final plans for three-sided precast concrete structures shall identify the size (span x rise), length, and skew angle of the bridge.

3. Final plans shall include all geometric dimensions and a detailed design for the three-sided precast structure, all cast-in-place foundation units and cast-in-place or precast wingwalls and headwalls.

4. Final plans shall include the pay item Three-Sided Precast Concrete Structure and applicable pay items for the remainder of the substructure elements.

5. Final plans shall be submitted along with all pertinent special provisions to the BOS for review and approval.

In addition to foundation type, the wingwall type shall be provided on the preliminary and final plans. Similar to precast boxes, a wingwall design shall be provided which is supported independently from the three-sided structure. The restrictions on the use of cast-in-place or precast wings and headwalls shall be based on site conditions and the preferences of the Owner. These restrictions shall be noted on the preliminary and final plans.

36.13.4 Foundation Requirements

Precast and cast-in-place three-sided structures that are utilized in pedestrian or cattle underpasses can be supported on continuous spread or pile supported footings. Precast and cast-in-place three-sided structures that are utilized in waterway applications shall be supported on piling to prevent scour.

The footing should be kept level if possible. If the stream grade prohibits a level footing, the wingwall footings should be laid out to be constructed on the same plane as the structure footings. Continuity shall be established between the structural unit footing and the wingwall footing.

The allowable soil bearing pressure should be shown on the plans. Weak soil conditions could require pile foundations. If the footing is on piling, the nominal driving resistance should be shown. Where a pile footing is required, the type and size of pile and the required pile spacing, and which piles are to be battered, should be shown on the plans.

The geotechnical engineer should provide planning and design recommendations to determine the most cost effective and feasible foundation treatment to be used on the preliminary plans.

36.13.5 Precast Versus Cast-in-Place Wingwalls and Headwalls

The specifications for three-sided precast concrete structures permits the contractor to substitute cast-in-place for precast wingwalls and headwalls, and vice versa when cast-in-place is specified unless prohibited on the plans. Three-sided structures should be provided with adequate foundation support to satisfy the design assumptions permitting their relatively thin concrete section. These foundations are designed and provided in the plans. Spread footing foundations are most commonly used since they prove cost effective when rock or scour resistant soils are present with adequate bearing and sliding resistance. The use of precast spread footings shall be controlled by the planner and shall only be allowed when soil conditions permit and shall not be allowed to bear directly on rock or when rock is within 2 feet of the bottom of the proposed footing. When lower strength soils are present, or scour depths become large, a pile supported footing shall be used. The lateral loading design of the foundation is important because deflection of the pile or footing should not exceed the manufacturers' recommendations to preclude cracks developing.

36.14 Design Example

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E36-1 Twin Cell Box Culvert LRFD

This example shows design calculations for a twin cell box culvert. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

E36-1.1 Design Criteria

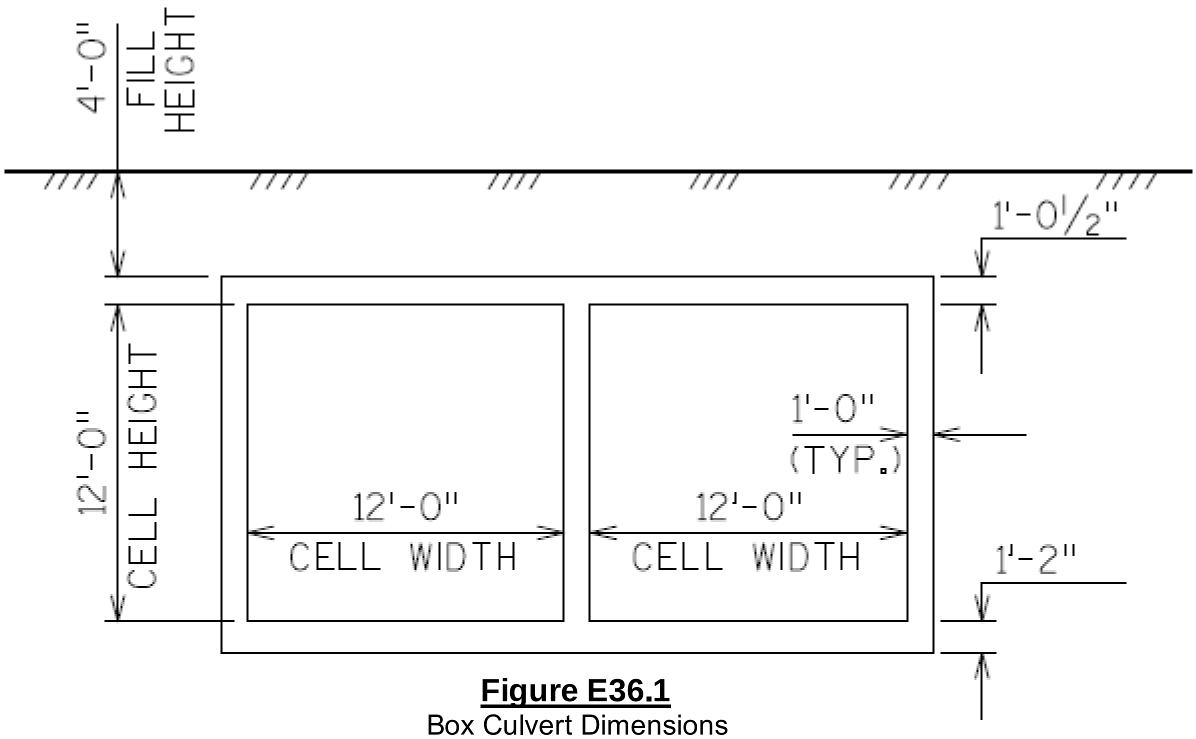


Figure E36.1
Box Culvert Dimensions

$NC = 2$	number of cells
$Ht = 12.0$	cell height, ft
$W_1 = 12.0$	cell 1 width, ft
$W_2 = 12.0$	cell 2 width, ft
$L = 134.0$	culvert length, ft
$t_{ts} = 12.5$	top slab thickness, in
$t_{bs} = 14.0$	bottom slab thickness, in
$t_{win} = 12.0$	interior wall thickness, in
$t_{wex} = 12.0$	exterior wall thickness, in

$$H_{apron} := Ht + \frac{t_{ts}}{12}$$ apron wall height above floor, ft

$H_{apron} = 13.04$ ft.

$f'_c := 3.5$	culvert concrete strength, ksi
$f_y := 60$	reinforcement yield strength, ksi
$E_s := 29000$	modulus of elasticity of steel, ksi
$skew = 0.0$	skew angle, degrees
$H_s = 4.00$	depth of backfill above top edge of top slab, ft
$w_c := 0.150$	weight of concrete, kcf
$cover_{bot} := 3$	concrete cover (bottom of bottom slab), in
$cover := 2$	concrete cover (all other applications), in
$LS_{ht} := 2.2$	live load surcharge height, ft (See Sect. 36.4.4)

Resistance factors, reinforced concrete cast-in-place box structures, **LRFD [Table 12.5.5-1]**

$\phi_f := 0.9$	resistance factor for flexure
$\phi_v := 0.85$	resistance factor for shear

Calculate the span lengths for each cell (measured between centerlines of walls)

$$S_1 := W_1 + \frac{1}{12}\left(\frac{t_{win}}{2} + \frac{t_{wex}}{2}\right) \qquad \boxed{S_1 = 13.00} \text{ ft}$$

$$S_2 = W_2 + \frac{1}{12}\left(\frac{t_{wex}}{2} + \frac{t_{win}}{2}\right) \qquad \boxed{S_2 = 13.00} \text{ ft}$$

Verify that the box culvert dimensions fall within WisDOT's minimum dimension criteria. Per Sect. 36.2, the minimum size for pedestrian underpasses is 8 feet high by 5 feet wide. The minimum size for cattle underpass is 6 feet high by 5 feet wide. A minimum height of 5 feet is desirable for cleanout purposes.

Does the culvert meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Verify that the slab and wall thicknesses fall within WisDOT's minimum dimension criteria. Per Sect. 36.5, the minimum thickness of the top and bottom slab is 6.5 inches. Per Sect. 36.5 [Table 36.5-1], the minimum wall thickness varies with respect to cell height and apron wall height.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

Do the slab and wall thicknesses meet the minimum dimension criteria? $\boxed{check = "OK"}$

Since this example has more than 2.0 feet of fill, edge beams are not req'd, **LRFD [C12.11.2.1]**

E36-1.2 Modulus of Elasticity of Concrete Material

Per Sect. 36.2.1, use $f`_c$ = 3.5 ksi for culverts. Calculate value of E_C per **LRFD [C5.4.2.4]**:

$\boxed{K_1 := 1}$ $E_{c\_calc} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ $\boxed{E_{c\_calc} = 3586.616}$ ksi

$E_c := 3600$ ksi modulus of elasticity of concrete, per Sect. 9.2

E36-1.3 Loads

$\gamma_s := 0.120$ unit weight of soil, kcf

Per Sect. 36.5, a haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Minimum haunch depth and length is 6 inches. Haunch depth is increased in 3 inch increments. For the first iteration, assume there are no haunches.

$h_{hau} := 0.0$ haunch height, in

$l_{hau} := 0.0$ haunch length, in

$wt_{hau} = 0.0$ weight of one haunch, kip

E36-1.3.1 Dead Loads

Dead Load (DC):

top slab dead load:

$$w_{dlts} := w_c \cdot \frac{t_{ts}}{12} \cdot 1 \qquad \boxed{w_{dlts} = 0.156} \text{ klf}$$

bottom slab dead load:

$$w_{dlbs} := w_c \cdot \frac{t_{bs}}{12} \cdot 1 \qquad \boxed{w_{dlbs} = 0.175} \text{ klf}$$

Wearing Surface (DW):

Per Sect. 36.4.2, the weight of the future wearing surface is zero if there is any fill depth over the culvert. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 0.020 ksf.

$w_{ws} = 0.000$ weight of future wearing surface, ksf

Vertical Earth Load (EV):

Calculate the modification of earth loads for soil-structure interaction per **LRFD [12.11.2.2]**. Per the policy item in Sect. 36.4.3, embankment installations are always assumed.

Installation_Type = "Embankment"

$\gamma_s = 0.120$ unit weight of soil, kcf

$B_c = 27.00$ outside width of culvert, ft
(measured between outside faces of exterior walls)

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

Calculate the soil-structure interaction factor for embankment installations:

$$F_e := 1 + 0.20 \cdot \frac{H_s}{B_c} \qquad \boxed{F_e = 1.03}$$

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section:

$$\boxed{F_e = 1.03}$$

Calculate the total unfactored earth load:

$W_E := F_e \cdot \gamma_s \cdot B_c \cdot H_s$ $\boxed{W_E = 13.34}$ klf

Distrubute the total unfactored earth load to be evenly distributed across the top of the culvert:

$w_{sv} := \frac{W_E}{B_c}$ $\boxed{w_{sv} = 0.494}$

Horizontal Earth Load (EH):

Soil horizontal earth load (magnitude at bottom and top of wall): **LRFD [3.11.5.1]**

$k_o := 0.5$ coefficient of at rest lateral earth pressure per Sect. 36.4.3

$\gamma_s = 0.120$ unit weight of soil, kcf

$w_{sh\_bot} := k_o \cdot \gamma_s \cdot \left(Ht + \frac{t_{ts}}{12} + \frac{t_{bs}}{12} + H_s\right) \cdot 1$ $\boxed{w_{sh\_bot} = 1.09}$ klf

$w_{sh\_top} := k_o \cdot \gamma_s \cdot (H_s) \cdot 1$ $\boxed{w_{sh\_top} = 0.24}$ klf

Live Load Surcharge (LS):

Soil live load surcharge: **LRFD [3.11.6.4]**

$k_o = 0.5$ coefficient of lateral earth pressure

$\gamma_s = 0.120$ unit weight of soil, kcf

$LS_{ht} = 2.2$ live load surcharge height per Sect. 36.4.4, ft

$w_{sll} := k_o \cdot \gamma_s \cdot LS_{ht} \cdot 1$ $\boxed{w_{sll} = 0.13}$ klf

E36-1.3.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = design truck (no lane) **LRFD [3.6.1.3.3]**

design tandem (no lane)

For the Wisconsin Standard Permit Vehicle (Wis-SPV) Check:

The Wis-SPV vehicle is to be checked during the design phase to make sure it can carry a minimum vehicle load of 190 kips. See Section 36.1.3 of the Bridge Manual for requirements pertaining to the Wis-SPV vehicle check.

E36-1.4 Live Load Distribution

Live loads are distributed over an equivalent area, with distribution components both parallel and perpendicular to the span, as calculated below. Per **LRFD [3.6.1.3.3]**, the live loads to be placed on these widths are axle loads (i.e., two lines of wheels) without the lane load. The equivalent distribution width applies for both live load moment and shear.

E36-1.5 Equivalent Strip Widths for Box Culverts

The calculations for depths of fill less than 2.0 ft, per **LRFD [4.6.2.10]** are not required for this example. The calcuations are shown for illustration purposes only.

The calculations below follow **LRFD [4.6.2.10.2]** - Case 1: Traffic Travels Parallel to Span. If traffic travels perpendicular to the span, follow **LRFD [4.6.2.10.3]** - Case 2: Traffic Travels Perpendicular to Span, which states to follow **LRFD [4.6.2.1]**.

Per **LRFD [4.6.2.10.2]**, when traffic travels primarily parallel to the span, culverts shall be analyzed for a single loaded lane with a single lane multiple presence factor (mpf).

Therefore, $mpf = 1.2$

Perpendicular to the span:

It is conservative to use the largest distribution factor from each span of the structure across the entire length of the culvert. Therefore, use the smallest span to calculate the smallest strip width. That strip width will provide the largest distribution factor.

$S := \min(W_1, W_2)$ clear span, ft $\boxed{S = 12.00}$ ft

The equivalent distribution width perpendicular to the span is:

$$E_{perp} := \frac{1}{12} \cdot (96 + 1.44 \cdot S) \qquad \boxed{E_{perp} = 9.44} \text{ ft}$$

Parallel to the span:

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

$L_T := 10$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$LLDF = 1.15$ live load distribution factor. From **LRFD [4.6.2.10.2]**, LLDF = 1.15 as specified in **LRFD [Table 3.6.1.2.6a-1]** for select granular backfill

The equivalent distribution width parallel to the span is:

$$E_{parallel} := \frac{1}{12} \cdot \left(L_T + LLDF \cdot H_s \cdot 12\right) \qquad \boxed{E_{parallel} = 5.43} \text{ ft}$$

The equivalent distribution widths parallel and perpendicular to the span create an area that the axial load shall be distributed over. The equivalent area is:

$$E_{area} := E_{perp} \cdot E_{parallel} \qquad \boxed{E_{area} = 51.29} \text{ ft}^2$$

For depths of fill 2.0 ft. or greater calculate the size of the rectangular area that the wheels are considered to be uniformly distributed over, per Sect. 36.4.6.2.

$L_T = 10.0$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$W_T := 20$ width of tire contact area, in **LRFD [3.6.1.2.5]**

The length and width of the equivalent area for 1 wheel are: **LRFD [3.6.1.2.6b]**

$L_{eq\_i} := L_T + LLDF \cdot H_s \cdot 12$ $\boxed{L_{eq\_i} = 65.20}$ in

$W_{eq\_i} := W_T + LLDF \cdot H_s \cdot 12 + 0.06 \cdot \max(W_1, W_2) 12$ $\boxed{W_{eq\_i} = 83.84}$ in

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area, **LRFD [3.6.1.2.6a]**.

Check if the areas overlap = "Yes, the areas overlap" therefore, use the following length and width values for the equivalent area for 1 wheel:

	Front and Rear Wheels:		Center Wheel:	
Length	$L_{eq13} = 65.2$	in	$L_{eq2} = 65.2$	in
Width	$W_{eq13} = 77.9$	in	$W_{eq2} = 77.9$	in
Area	$A_{eq13} = 5080.4$	in^2	$A_{eq2} = 5080.4$	in^2

Per **LRFD [3.6.1.2.2]**, the weights of the design truck wheels are below. (Note that one axle load is equal to two wheel loads.)

$W_{wheel1i} := 4000$ front wheel weight, lbs

$W_{wheel23i} := 16000$ center and rear wheel weights, lbs

The effect of single and multiple lanes shall be considered. For this problem, a single lane with the single lane multiple presence factor (mpf) governs. Applying the single lane multiple presence factor:

$W_{wheel1} := mpf \cdot W_{wheel1i}$ $W_{wheel1} = 4800.00$ lbs $mpf = 1.20$

$W_{wheel23} := mpf \cdot W_{wheel23i}$ $W_{wheel23} = 19200.00$ lbs

For single-span culverts, the effects of the live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects of the live load may be neglected where the depth of fill exceeds the distance between faces of endwalls, **LRFD [3.6.1.2.6a]**.
Note: The wheel pressure values shown here are for the 14'-0" variable axle spacing of the design truck, which controls over the design tandem for this example. In general, all variable axle spacings of the design truck and the design tandem must be investigated to account for the maximum response. Dividing the wheel loads (incl. mpf) by the equivalent area gives:

$LL1 = 0.94$ live load pressure (front wheel), psi

$LL2 = 3.78$ live load pressure (center wheel), psi

$LL3 = 3.78$ live load pressure (rear wheel), psi

E36-1.6 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in Chapter 36 of this manual and as indicated below.

E36-1.6.1 Load Factors

From **LRFD [Table 3.4.1-1]** and **LRFD [Table 3.4.1-2]**:

Per the policy item in Sect. 36.4.3: Assume box culverts are closed, rigid frames for Strength 1 (EV-factor). Assume active earth pressure to be conservative for Strength 1 (EH-factor).

	Strength 1	Service 1
DC	$\gamma st_{DCmax} := 1.25$	$\gamma s1_{DC} := 1.0$
	$\gamma st_{DCmin} := 0.9$	
DW	$\gamma st_{DWmax} := 1.5$	$\gamma s1_{DW} := 1.0$
	$\gamma st_{DWmin} := 0.65$	
EV	$\gamma st_{EVmax} := 1.35$	$\gamma s1_{EV} := 1.0$
	$\gamma st_{EVmin} := 0.9$	
EH	$\gamma st_{EHmax} := 1.50$	$\gamma s1_{EH} := 1.0$
	$\gamma st_{EHmin} := 0.5$ **LRFD [3.11.7]**	
LS	$\gamma st_{LSmax} := 1.75$	$\gamma s1_{LS} := 1.0$
	$\gamma st_{LSmin} := 0$	
LL	$\gamma st_{LL} := 1.75$	$\gamma s1_{LL} := 1.0$

Dynamic Load Allowance (IM) is applied to the truck and tandem. From **LRFD [3.6.2.2]**, IM of buried components varies with depth of cover above the structure and is calculated as:

$IM := 33 \cdot (1.0 - 0.125 \cdot H_S)$ (where H_S is in feet) $IM = 16.50$

If IM is less than 0, use IM = 0 $\boxed{IM = 16.50}$

E36-1.6.2 Dead Load Moments and Shears

The unfactored dead load moments and shears for each component are listed below (values are per 1-foot width and are in kip-ft and kip, respectively):

Exterior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-1.52	-1.44	-5.14	-1.01	0.00
0.1	-1.42	-1.54	-0.12	-0.14	0.00
0.2	-1.31	-1.63	3.53	0.55	0.00
0.3	-1.21	-1.73	5.92	1.04	0.00
0.4	-1.10	-1.82	7.14	1.34	0.00
0.5	-1.00	-1.91	7.30	1.46	0.00
0.6	-0.89	-2.01	6.51	1.38	0.00
0.7	-0.79	-2.10	4.87	1.12	0.00
0.8	-0.68	-2.19	2.49	0.66	0.00
0.9	-0.58	-2.29	-0.54	0.01	0.00
1.0	-0.48	-2.38	-4.11	-0.82	0.00

Interior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

Top Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.04	-1.14	-5.47	-1.18	0.00
0.1	0.73	1.45	-4.67	-1.00	0.00
0.2	1.27	3.32	-3.87	-0.83	0.00
0.3	1.60	4.48	-3.07	-0.66	0.00
0.4	1.69	4.93	-2.27	-0.49	0.00
0.5	1.56	4.67	-1.47	-0.32	0.00
0.6	1.21	3.69	-0.67	-0.15	0.00
0.7	0.63	2.01	0.13	0.03	0.00
0.8	-0.18	-0.39	0.93	0.20	0.00
0.9	-1.21	-3.50	1.72	0.37	0.00
1.0	-2.46	-7.32	2.52	0.54	0.00

Bottom Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.60	-0.17	-7.63	-1.42	0.00
0.1	1.36	2.26	-6.51	-1.21	0.00
0.2	2.76	3.98	-5.39	-1.00	0.00
0.3	3.61	4.99	-4.27	-0.79	0.00
0.4	3.91	5.29	-3.15	-0.59	0.00
0.5	3.65	4.87	-2.03	-0.38	0.00
0.6	2.85	3.75	-0.90	-0.17	0.00
0.7	1.49	1.91	0.22	0.04	0.00
0.8	-0.42	-0.64	1.34	0.25	0.00
0.9	-2.88	-3.90	2.46	0.46	0.00
1.0	-5.89	-7.88	3.58	0.67	0.00

Exterior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.09	-0.08	4.78	0.73	0.00
0.1	0.09	-0.08	3.60	0.59	0.00
0.2	0.09	-0.08	2.50	0.45	0.00
0.3	0.09	-0.08	1.49	0.30	0.00
0.4	0.09	-0.08	0.56	0.16	0.00
0.5	0.09	-0.08	-0.27	0.01	0.00
0.6	0.09	-0.08	-1.03	-0.13	0.00
0.7	0.09	-0.08	-1.69	-0.27	0.00
0.8	0.09	-0.08	-2.27	-0.42	0.00
0.9	0.09	-0.08	-2.76	-0.56	0.00
1.0	0.09	-0.08	-3.17	-0.71	0.00

Interior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

Top Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.74	2.45	0.67	0.13	0.00
0.1	0.55	1.86	0.67	0.13	0.00
0.2	0.36	1.26	0.67	0.13	0.00
0.3	0.17	0.67	0.67	0.13	0.00
0.4	-0.01	0.08	0.67	0.13	0.00
0.5	-0.20	-0.52	0.67	0.13	0.00
0.6	-0.39	-1.11	0.67	0.13	0.00
0.7	-0.58	-1.70	0.67	0.13	0.00
0.8	-0.76	-2.30	0.67	0.13	0.00
0.9	-0.95	-2.89	0.67	0.13	0.00
1.0	-1.14	-3.48	0.67	0.13	0.00

Bottom Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	1.86	2.32	0.94	0.16	0.00
0.1	1.40	1.73	0.94	0.16	0.00
0.2	0.94	1.14	0.94	0.16	0.00
0.3	0.48	0.54	0.94	0.16	0.00
0.4	0.02	-0.05	0.94	0.16	0.00
0.5	-0.44	-0.64	0.94	0.16	0.00
0.6	-0.90	-1.24	0.94	0.16	0.00
0.7	-1.36	-1.83	0.94	0.16	0.00
0.8	-1.82	-2.42	0.94	0.16	0.00
0.9	-2.28	-3.01	0.94	0.16	0.00
1.0	-2.74	-3.61	0.94	0.16	0.00

The DC values are the component dead loads and include the self weight of the culvert and haunch (if applicable).

The DW values are the dead loads from the future wearing surface (DW values occur only if there is no fill on the culvert).

The EV values are the vertical earth loads from the fill on top of the box culvert.

The EH values are the horizontal earth loads from the fill on the sides of the box culvert.

The LS values are the live load surcharge loads (assuming $LS_{ht} = 2.2$ feet of surcharge)

E36-1.6.3 Live Load Moments and Shears

The unfactored live load load moments and shears (per lane including impact) are listed below (values are in kip-ft and kips, respectively). A separate analysis run will be required if results without impact are desired.

Exterior Wall
Unfactored Live Load Moments (kip-ft)

Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.73	-1.74	0.74	-1.77
0.1	0.67	-1.70	0.69	-1.92
0.2	0.61	-1.67	0.65	-2.07
0.3	0.55	-1.65	0.62	-2.21
0.4	0.48	-1.68	0.60	-2.36
0.5	0.42	-1.82	0.58	-2.51
0.6	0.37	-1.97	0.56	-2.69
0.7	0.41	-2.12	0.56	-2.86
0.8	0.47	-2.28	0.61	-3.04
0.9	0.55	-2.44	0.68	-3.21
1.0	0.65	-2.61	0.77	-3.39

Interior Wall
Unfactored Live Load Moments (kip-ft)

Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.99	-0.99	0.88	-0.88
0.1	0.93	-0.93	0.99	-0.99
0.2	0.92	-0.92	1.12	-1.12
0.3	0.90	-0.90	1.25	-1.25
0.4	0.90	-0.90	1.38	-1.38
0.5	1.08	-1.08	1.54	-1.53
0.6	1.27	-1.27	1.74	-1.74
0.7	1.47	-1.47	1.99	-1.99
0.8	1.69	-1.69	2.24	-2.24
0.9	1.92	-1.92	2.50	-2.50
1.0	2.17	-2.17	2.75	-2.75

Top Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.81	-1.76	0.65	-2.16
0.1	2.24	-0.34	1.83	-0.20
0.2	3.81	-0.27	4.23	-0.32
0.3	5.06	-0.49	5.92	-0.66
0.4	5.71	-0.75	6.78	-1.04
0.5	5.76	-1.04	6.90	-1.43
0.6	5.22	-1.34	6.21	-1.82
0.7	4.13	-1.64	4.74	-2.22
0.8	2.56	-1.96	2.54	-2.62
0.9	0.86	-3.59	0.76	-3.02
1.0	0.07	-5.89	0.06	-4.81

Bottom Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.46	-0.67	0.40	-0.35
0.1	1.72	-0.29	2.52	-0.32
0.2	3.30	-0.76	4.46	-0.78
0.3	4.25	-1.06	5.63	-1.09
0.4	4.60	-1.24	6.06	-1.30
0.5	4.39	-1.34	5.82	-1.45
0.6	3.68	-1.39	4.96	-1.62
0.7	2.56	-1.46	3.55	-1.86
0.8	1.18	-1.57	1.62	-2.23
0.9	0.00	-2.40	0.00	-2.79
1.0	0.00	-4.90	0.00	-3.75

Exterior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.11	-0.19	0.09	-0.16
0.1	0.11	-0.19	0.09	-0.16
0.2	0.11	-0.19	0.09	-0.16
0.3	0.11	-0.19	0.09	-0.16
0.4	0.11	-0.19	0.09	-0.16
0.5	0.11	-0.19	0.09	-0.16
0.6	0.11	-0.19	0.09	-0.16
0.7	0.11	-0.19	0.09	-0.16
0.8	0.11	-0.19	0.09	-0.16
0.9	0.11	-0.19	0.09	-0.16
1.0	0.11	-0.19	0.09	-0.16

Interior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.23	-0.23	0.21	-0.21
0.1	0.23	-0.23	0.21	-0.21
0.2	0.23	-0.23	0.21	-0.21
0.3	0.23	-0.23	0.21	-0.21
0.4	0.23	-0.23	0.21	-0.21
0.5	0.23	-0.23	0.21	-0.21
0.6	0.23	-0.23	0.21	-0.21
0.7	0.23	-0.23	0.21	-0.21
0.8	0.23	-0.23	0.21	-0.21
0.9	0.23	-0.23	0.21	-0.21
1.0	0.23	-0.23	0.21	-0.21

Top Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.71	-0.26	3.24	-0.33
0.1	2.33	-0.33	2.67	-0.33
0.2	1.95	-0.47	2.11	-0.33
0.3	1.56	-0.69	1.59	-0.39
0.4	1.19	-1.00	1.14	-0.67
0.5	0.85	-1.37	0.78	-1.03
0.6	0.54	-1.74	0.49	-1.46
0.7	0.30	-2.10	0.27	-1.97
0.8	0.14	-2.44	0.12	-2.54
0.9	0.04	-2.76	0.04	-3.11
1.0	0.00	-3.05	0.00	-3.66

Bottom Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.19	-0.68	2.69	-0.68
0.1	1.61	-0.48	1.97	-0.48
0.2	1.06	-0.32	1.29	-0.32
0.3	0.54	-0.19	0.66	-0.21
0.4	0.06	-0.11	0.07	-0.14
0.5	0.01	-0.45	0.00	-0.46
0.6	0.02	-0.90	0.02	-0.96
0.7	0.02	-1.33	0.02	-1.40
0.8	0.01	-1.74	0.01	-1.80
0.9	0.00	-2.12	0.00	-2.15
1.0	0.00	-2.48	0.00	-2.46

E36-1.6.4 Factored Moments

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored moments for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Moments

$$M_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot M_{DC} + \gamma st_{DW} \cdot M_{DW} + \gamma st_{EV} \cdot M_{EV} + \gamma st_{EH} \cdot M_{EH} + \gamma st_{LS} \cdot M_{LS} + \gamma st_{LL} \cdot M_{LL}\right)$$

Corner Bars	$Mstr1_{CB} = 17.34$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Mstr1_{PTS} = 19.59$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Mstr1_{PBS} = 21.05$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Mstr1_{NTS} = 22.00$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Mstr1_{NBS} = 24.77$	kip-ft	(negative moment)
Exterior Wall Bars	$Mstr1_{XW} = 11.90$	kip-ft	(positive moment)
Interior Wall Bars	$Mstr1_{IW} = 4.82$	kip-ft	(positive moment)

Service 1 Moments

$$M_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot M_{DC} + \gamma s1_{DW} \cdot M_{DW} + \gamma s1_{EV} \cdot M_{EV} + \gamma s1_{EH} \cdot M_{EH} + \gamma s1_{LS} \cdot M_{LS} + \gamma s1_{LL} \cdot M_{LL}\right)$$

Corner Bars	$Ms1_{CB} = 11.18$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Ms1_{PTS} = 11.66$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Ms1_{PBS} = 12.32$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Ms1_{NTS} = 13.15$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Ms1_{NBS} = 15.08$	kip-ft	(negative moment)
Exterior Wall Bars	$Ms1_{XW} = 6.43$	kip-ft	(positive moment)
Interior Wall Bars	$Ms1_{IW} = 2.75$	kip-ft	(positive moment)

E36-1.7 Design Reinforcement Bars

Design of the corner bars is illustrated below. Calculations for bars in other locations are similar.

Design Criteria:

For corner bars, use the controlling thickness between the slab and wall. The height of the concrete design section is:

$h := \min(t_{ts}, t_{bs}, t_{wex})$ $\boxed{h = 12.00}$ in

Use a 1'-0" design width:

$b := 12.0$ width of the concrete design section, in

$cover = 2.0$ concrete cover, in Note: The calculations here use 2" cover for the top slab and walls. Use 3" cover for the bottom of the bottom slab (not shown here).

$Mstr1_{CB} = 17.34$ design strength moment, kip-ft

$Ms1_{CB} = 11.18$ design service moment, kip-ft

$f_s := f_y$ reinforcement yield strength, ksi $f_y = 60.00$ ksi

$Bar_{No} := 5$ assume #5 bars (for d_s calculation)

$Bar_D(Bar_{No}) = 0.63$ bar diameter, in

Calculate the estimated distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement. **LRFD [5.7.3.2.2]**

$$d_{s\_i} := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_{s\_i} = 9.69} \text{ in}$$

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ per **LRFD [Table 12.5.5-1]**.

Calculate the coefficient of resistance:

$$R_n := \frac{Mstr1_{CB} \cdot 12}{\phi_f \cdot b \cdot d_{s\_i}^{\ 2}} \qquad \boxed{R_n = 0.21} \text{ ksi}$$

Calculate the reinforcement ratio:

$$\rho := 0.85 \cdot \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1.0 - \frac{2 \cdot R_n}{0.85 \cdot f'_c}}\right) \qquad \boxed{\rho = 0.0035}$$

Calculate the required area of steel:

$A_{s\_req'd} := \rho \cdot b \cdot d_{s\_i}$ $\boxed{A_{s\_req'd} = 0.41}$ in^2

Given the required area of steel of $A_{s\_req'd} = 0.41$, try #5 bars at 7.5" spacing:

$Bar_{No} := 5$ bar size

$spacing := 7.0$ bar spacing, in

The area of one reinforcing bar is:

$A_{s\_1bar} := Bar_A(Bar_{No})$ $\boxed{A_{s\_1bar} = 0.31}$ in^2

Calculate the area of steel in a 1'-0" width

$$A_s := \frac{A_{s\_1bar}}{\frac{spacing}{12}}$$ $\boxed{A_s = 0.53}$ in^2

Check that the area of steel provided is larger than the required area of steel

Is $A_s = 0.53\ in^2 \geq A_{s\_req'd} = 0.41\ in^2$ $\boxed{check = \text{"OK"}}$

Recalculate d_c and d_s based on the actual bar size used.

$$d_c := cover + \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_c = 2.31}$ in

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_s = 9.69}$ in

Per **LRFD [5.7.2.2]**, The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65. The factor α_1 shall be taken as 0.85 for concrete strength not exceeding 10.0 ksi.

$\boxed{\beta_1 = 0.85}$ $\boxed{\alpha_1 = 0.85}$

Per **LRFD [5.7.2.1]**, if $\frac{c}{d_s} \leq 0.6$ (for f_y = 60 ksi) then reinforcement has yielded and the assumption is correct.

"c" is defined as the distance between the neutral axis and the compression face (inches).

$$c := \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$ $\boxed{c = 1.05}$ in

Check that the reinforcement will yield:

Is $\frac{c}{d_s} = 0.11 \leq 0.6?$ $\boxed{check = \text{"OK"}}$

therefore, the reinforcement will yield

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.7.3.2.3]**:

$a := \beta_1 \cdot c$ $\boxed{a = 0.89}$ in

$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right) \frac{1}{12}\right]$ $\boxed{M_n = 24.6}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 22.1}$ kip-ft

The required capacity:

Corner Moment $\boxed{Mstr1_{CB} = 17.3}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]**:

$b = 12.0$ in width of the concrete design section, in

$h = 12.0$ in height of the concrete design section, in

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $f_r = 0.45$ ksi

$I_g := \frac{1}{12} \cdot b \cdot h^3$ gross moment of inertia, in^4 $I_g = 1728.00$ in^4

$\frac{h}{2} = 6.0$ distance from the neutral axis to the extreme element

$S_c := \frac{I_g}{\frac{h}{2}}$ section modulus, in^3 $S_c = 288.00$ in^3

The corresponding cracking moment is:

$M_{cr} = \gamma_3 (\gamma_1 \cdot f_r) S_c$ therefore, $M_{cr} = 1.1 (f_r) S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement <u>for A615, Grade 60 reinforcement</u>

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CB} = 23.1}$ kip-ft

Is $M_r = 22.1$ kip-ft greater than the lesser of M_{cr} and $1.33 {}^* M_{str}$? $\boxed{check = \text{"OK"}}$

Per **LRFD [5.7.3.4]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.0$ for Class 1 exposure condition

$h = 12.0$ height of the concrete design section, in

Calculate the ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face:

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.34}$$

Calculate the reinforcement ratio:

$$\rho := \frac{A_s}{b \cdot d_s} \qquad \boxed{\rho = 0.0046}$$

Calculate the modular ratio:

$$N := \frac{E_s}{E_c} \qquad \boxed{N = 8.06}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi). The moment arm used in the equation below to calculate f_{ss} is: (j) $(h\text{-}d_c)$

$$k := \sqrt{(\rho \cdot N)^2 + (2 \cdot \rho \cdot N)} - \rho \cdot N \qquad \boxed{k = 0.2370}$$

$$j := 1 - \frac{k}{3} \qquad \boxed{j = 0.9210}$$

$Ms1_{CB} = 11.18$ service moment, kip-ft

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \le 0.6\, f_y \qquad \boxed{f_{ss} = 28.29} \text{ ksi} \quad \le 0.6\, f_y \text{ O.K.}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad s_{max1} = 13.83 \text{ in}$$

$$s_{max2} := \min(1.5 \cdot h, 18) \qquad s_{max2} = 18.00 \text{ in}$$

$$s_{max} := \min(s_{max1}, s_{max2}) \qquad \boxed{s_{max} = 13.83} \text{ in}$$

Check that the provided spacing is less than the maximum allowable spacing

Is $spacing = 7.00$ in $\leq$ $s_{max} = 13.83$ in $\boxed{check = \text{"OK"}}$

Calculate the minimum spacing requirements per **LRFD [5.10.3.1]**. The clear distance between parallel bars in a layer shall not be less than:

$$S_{min1} := 1.5 \cdot Bar_D(Bar_{No}) \qquad S_{min1} = 0.94 \text{ in}$$

$$S_{min2} := 1.5 \cdot 1.5 \quad \text{(maximum aggregate size = 1.5 inches)} \qquad S_{min2} = 2.25 \text{ in}$$

$$S_{min3} := 1.5 \text{ in}$$

Is $spacing = 7.00$ in $\geq$ all minimum spacing requirements? $\boxed{check = \text{"OK"}}$

E36-1.8 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceding sections.

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8]**

$$A_s \geq \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \qquad \text{and} \qquad 0.11 \leq A_s \leq 0.60$$

Where:

A_s = area of reinforcement in each direction and each face $\left(\frac{in^2}{ft}\right)$

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Check the minimum required temperature and shrinkage reinforcement, #4 bars at 15", in the thickest section. For the given cross section, the values for the corner bar design are:

$$A_{s\_4\_at\_15} := \frac{Bar_A(4)}{1.25} \qquad \boxed{A_{s\_4\_at\_15} = 0.16} \ \frac{in^2}{ft}$$

$b_{TS} := \max(t_{ts}, t_{bs}, t_{wex})$ $\boxed{b_{TS} = 14.0}$ in

$h_{TS} := 12(W_1 + W_2) + 2 \cdot t_{wex} + t_{win}$ $\boxed{h_{TS} = 324.0}$ in

$f_y = 60.00$ ksi

For each face, the required area of steel is:

$$A_{s\_TS} := \frac{1.30 \cdot (b_{TS}) \cdot h_{TS}}{2 \cdot (b_{TS} + h_{TS}) \cdot f_y} \qquad A_{s\_TS} = 0.15 \quad \frac{in^2}{ft}$$

is $A_{s\_4\_at\_15} = 0.16$ in2 $\geq$ $A_{s\_TS} = 0.15$ in^2 ? $\boxed{check = "OK"}$

is $0.11 < A_{s\_4\_at\_15} < 0.60$? $\boxed{check = "OK"}$

Per **LRFD [5.10.8]**, the shrinkage and temperature reinforcement shall not be spaced farther apart than:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in for walls and footings greater than 18.0 in. thick
- 12.0 in for other components greater than 36.0 in. thick

$s_{max3} = 18.00$ in

Per **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the member or 18.0 in.

$s_{max4} = 18.00$ in

is the 15" spacing $\leq$ both maximum spacing requirements? $\boxed{check = "OK"}$

Note: The design of the bottom slab shrinkage and temperature bars is illistrated above. Shrinkage and temperature bars may be reduced or not required at other locations. See Section 36.6.8 and Standard 36.03 for additional information.

The results for the other bar locations are shown in the table below:

Results						
Location	ΦMn	$A_{S\ Req'd}$	$A_{S\ Actual}$	Bar Size	S_{max}	S_{actual}
Corner	22.1	0.48	0.53	5	13.8	7.0
Pos. Mom. Top Slab	21.8	0.49	0.50	5	13.0	7.5
Pos. Mom. Bot. Slab	28.9	0.54	0.57	5	18.0	6.5
Neg. Mom. Top Slab	23.3	0.50	0.53	5	12.1	7.0
Neg. Mom. Bot. Slab	28.4	0.54	0.62	5	13.4	6.0
Exterior Wall	16.9	0.37	0.40	4	18.0	6.0
Interior Wall	6.9	0.15	0.16	4	18.0	15.0

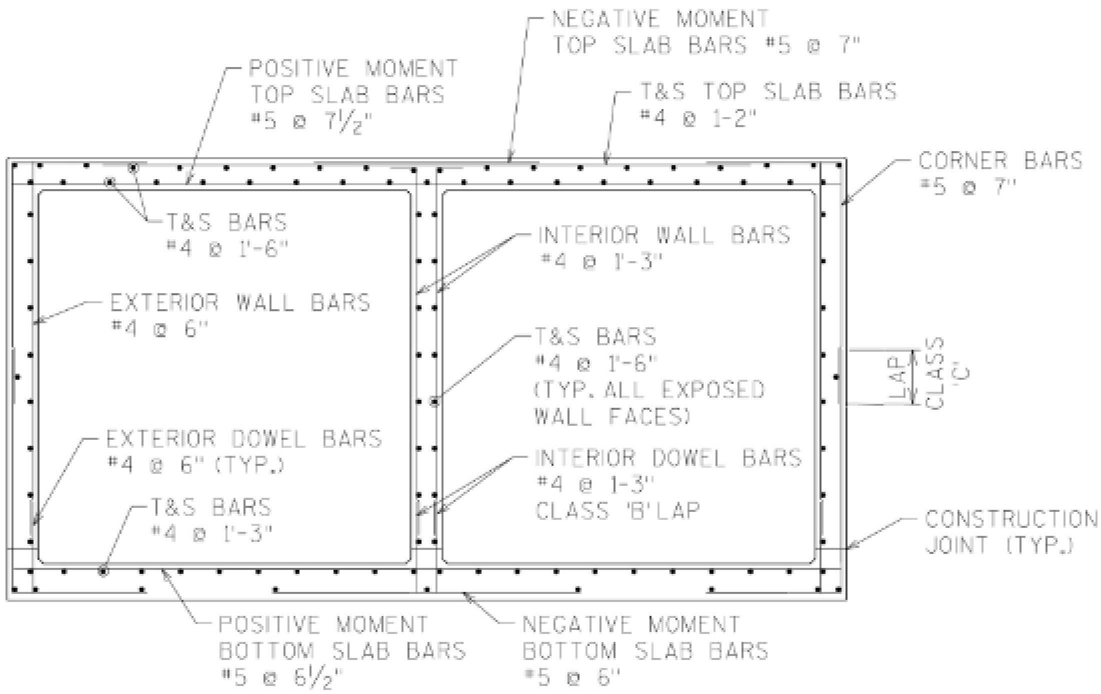
E36-1.9 Distribution Reinforcement

Per **LRFD [9.7.3.2]**, reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

Distribution steel is not required when the depth of fill over the slab exceeds 2 feet, **LRFD [5.14.4.1]**.

E36-1.10 Reinforcement Details

The reinforcement bar size and spacing required from the strength and serviceability calcuations above are shown below:



E36-1.11 Cutoff Locations

Determine the cutoff locations for the corner bars. Per Sect. 36.6.1, the distance "L" is computed from the maximum negative moment envelope for the top slab.

The cutoff lengths are in feet, measured from the inside face of the exterior wall.

Initial Cutoff Locations:

The initial cutoff locations are determined from the inflection points of the moment diagrams.

Corner Bars	$CutOff1_{CBH\_i} = 2.64$	$CutOff2_{CBH\_i} = 1.40$	Horizontal
		$CutOff2_{CBV\_i} = 2.08$	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS\_i} = 1.26$	$CutOff2_{PTS\_i} = 1.86$	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS\_i} = 1.27$	$CutOff2_{PBS\_i} = 1.97$	
Negative Moment Top Slab Bars	$CutOff1_{NTS\_i} = 8.63$	$CutOff2_{NTS\_i} = 10.32$	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS\_i} = 8.97$	$CutOff2_{NBS\_i} = 10.56$	

For the second cutoff location for each component, the following checks shall be completed:

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]**:

The required capacity at the second cutoff location (for the vertical leg of the corner bar):

$Mstr1_{CBV2} = 7.89$ strength moment at the second cutoff location, kip-ft

The usable capacity of the remaining bars is calculated as follows:

$$A_{s2} := \frac{A_s}{2} \qquad \boxed{A_{s2} = 0.27} \text{ in}^2$$

$$c2 := \frac{A_{s2} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \qquad \boxed{\beta_1 = 0.85} \quad \boxed{\alpha_1 = 0.85} \qquad \boxed{c2 = 0.53} \text{ in}$$

$$a2 := \beta_1 \cdot c2 \qquad \boxed{a2 = 0.45} \text{ in}$$

$$M_{n2} := \left[A_{s2} \cdot f_s \cdot \left(d_s - \frac{a2}{2}\right) \frac{1}{12}\right] \qquad \boxed{M_{n2} = 12.6} \text{ kip-ft}$$

$$M_{r2} := \phi_f \cdot M_{n2} \qquad \boxed{M_{r2} = 11.3} \text{ kip-ft}$$

Is $M_{r2} = 11.3$ kip-ft greater than the lesser of M_{cr} and 1.33*M_{str}? $\boxed{check = "OK"}$

$\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CBV2} = 10.5}$ kip-ft

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi).

$Ms1_{CBV2} = 3.43$ service moment at the second cutoff location, kip-ft

$$f_{ss2} := \frac{Ms1_{CBV2} \cdot 12}{A_{s2} \cdot (j) \cdot (h - d_c)}$$ $\boxed{f_{ss2} = 17.35}$ ksi

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max2\_1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss2}} - 2 \cdot d_c$$ $s_{max2\_1} = 25.47$ in

$s_{max2\_2} := s_{max2}$ $s_{max2\_2} = 18.00$ in

$s_{max} := \min(s_{max2\_1}, s_{max2\_2})$ $\boxed{s_{max} = 18.00}$ in

Check that the provided spacing (for half of the bars) is less than the maximum allowable spacing

$spacing2 := 2 \cdot spacing$ $spacing2 = 14.00$ in

Is $spacing2 = 14.00$ in $\leq$ $s_{max} = 18.00$ in $\boxed{check = "OK"}$

Extension Lengths:

The extension lengths for the corner bars are shown below. Calculations for other bars are similar.

Extension lengths for general reinforcement per **LRFD [5.11.1.2.1]**:

$$MaxDepth := \max\left(t_{ts} - cover, t_{wex} - cover, t_{bs} - cover_{bot}\right) \qquad MaxDepth = 11.00 \quad in$$

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

15 x bar diameter $\dfrac{15 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.78$ ft

1/20 times clear span $\dfrac{\max\left(W_1, W_2\right)}{20} = 0.60$ ft

The maximum of the values listed above:

$$ExtendLength\_gen_{CB} = 0.89 \quad ft$$

Extension lengths for negative moment reinforcement per **LRFD [5.11.1.2.3]**:

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

12 x bar diameter $\dfrac{12 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.63$ ft

0.0625 times clear span $0.0625 \max\left(W_1, W_2\right) = 0.75$ ft

The maximum of the values listed above:

$$ExtendLength\_neg_{CB} = 0.89 \quad ft$$

The development length:

$$DevLength_{CB} = 1.00 \quad ft$$

The extension lengths for general reinforcment for the other bars are:

Corner Bars	$ExtendLength\_gen_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength\_gen_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength\_gen_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength\_gen_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength\_gen_{NBS} = 0.97$	ft

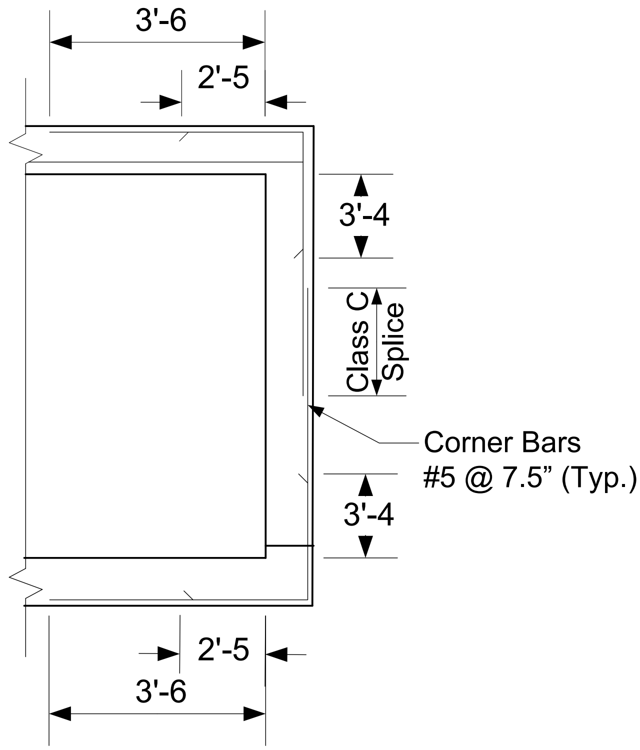
The extension lengths for negative moment reinforcment for the other bars are:

Corner Bars	$ExtendLength\_neg_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength\_neg_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength\_neg_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength\_neg_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength\_neg_{NBS} = 0.97$	ft

The final cutoff locations (measured from the inside face of the exterior wall) are:

Corner Bars	$CutOff1_{CBH} = 3.53$	$CutOff2_{CBH} = 2.29$	Horizontal
		$CutOff2_{CBV} = 2.97$	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PTS} = 1.02$	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PBS} = 1.00$	
Negative Moment Top Slab Bars	$CutOff1_{NTS} = 7.78$	$CutOff2_{NTS} = 9.47$	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS} = 7.99$	$CutOff2_{NBS} = 9.59$	

The cutoff locations for the corner bars are shown below. Other bars are similar.



E36-1.12 Shear Analysis

Analyze walls and slabs for shear

E36-1.12.1 Factored Shears

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored shears for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Shears

$$V_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot V_{DC} + \gamma st_{DW} \cdot V_{DW} + \gamma st_{EV} \cdot V_{EV} + \gamma st_{EH} \cdot V_{EH} + \gamma st_{LS} \cdot V_{LS} + \gamma st_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vstr1_{XW} = 8.69$	kip
Interior Wall	$Vstr1_{IW} = 0.40$	kip
Top Slab	$Vstr1_{TS} = 12.20$	kip
Bottom Slab	$Vstr1_{BS} = 12.16$	kip

Service 1 Shears

$$V_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot V_{DC} + \gamma s1_{DW} \cdot V_{DW} + \gamma s1_{EV} \cdot V_{EV} + \gamma s1_{EH} \cdot V_{EH} + \gamma s1_{LS} \cdot V_{LS} + \gamma s1_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vs1_{XW} = 5.64$	kip
Interior Wall	$Vs1_{IW} = 0.23$	kip
Top Slab	$Vs1_{TS} = 7.62$	kip
Bottom Slab	$Vs1_{BS} = 7.96$	kip

E36-1.12.2 Concrete Shear Resistance

Check that the nominal shear resistance, V_n, of the concrete in the top slab is adequate for shear without shear reinforcement per **LRFD [5.14.5.3]**.

$$V_n = V_c = \left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s \le 0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s$$

$f'_c = 3.5$ culvert concrete strength, ksi

$A_{s\_TS} = 0.15$ area of reinforcing steel in the design width, in²/ft width

$h := t_{ts}$ height of concrete design section, in $h = 12.50$ in

$\lambda = 1.0$ normal wgt. conc. **LRFD [5.4.2.8]**

Calculate d_s, the distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_s = 10.19}$ in

$V_u := Vstr1_{TS}$ $\boxed{V_u = 12.2}$ kips

$M_u = 264.01$ factored moment occurring simultaneously with V_u, kip-in

$b := 12$ design width, in

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$\frac{V_u \cdot d_s}{M_u}$ shall not be taken to be greater than 1.0 $\frac{V_u \cdot d_s}{M_u} = 0.47$ < 1.0 OK

$$V_{r1s} := \phi_v \cdot \left[\left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_{s\_TS}}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s\right]$$ $\boxed{V_{r1s} = 14.1}$ kips

but ≤ $V_{r2s} := \phi_v \cdot \left(0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s\right)$ $\boxed{V_{r2s} = 24.5}$ kips

$V_{rs} := \min(V_{r1s}, V_{r2s})$ $\boxed{V_{rs} = 14.1}$ kips

Check that the provided shear capacity is adequate:

Is $V_u = 12.2$ kip ≤ $V_{rs} = 14.1$ kip ? $\boxed{check = \text{"OK"}}$

Note: For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than: **LRFD[5.14.5.3]** $0.0948 \cdot \lambda \sqrt{f'_c}\ b \cdot d_s$

V_c for slabs simply supported need not be taken to be less than: $0.0791 \cdot \lambda \sqrt{f'_c}\ b \cdot d_s$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

LRFD [5.8] and **LRFD [5.13.3.6]** apply to slabs of box culverts with less than 2.0 ft of fill.

Check that the nominal shear resistance, V_n, of the concrete in the walls is adequate for shear without shear reinforcement per **LRFD [5.8.3.3]**. Calculations shown are for the exterior wall.

$$V_n = V_c = 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v \le 0.25 \cdot f'_c \cdot b_v \cdot d_v$$

$\beta := 2$ **LRFD [5.8.3.4.1]**

$f'_c = 3.5$ culvert concrete strength, ksi

$b_v := 12$ effective width, in

$h := t_{wex}$ height of concrete design section, in $h = 12.00$ in

λ = 1.0 normal wgt. conc. **LRFD [5.4.2.8]**

Distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 9.69} \ \text{in}$$

The effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; **LRFD [5.8.2.9]**

$$d_{v\_i} = d_s - \frac{a}{2}$$

from earlier calculations:

$\boxed{\beta_1 = 0.85}$

$\boxed{f_s = 60}$ ksi

$\boxed{A_{s\_XW} = 0.40}$ in²

The distance between the neutral axis and the compression face:

$$c := \frac{A_{s\_XW} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_v} \qquad \boxed{\beta_1 = 0.85} \ \boxed{\alpha_1 = 0.85} \qquad \boxed{c = 0.79} \ \text{in}$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 0.67} \ \text{in}$$

The effective shear depth:

$$d_{v\_i} := \left(d_s - \frac{a}{2}\right) \qquad \boxed{d_{v\_i} = 9.35}$$

d_v need not be taken to be less than the greater of 0.9 d_s or 0.72h (in.)

$$d_v := \max\left(d_{v\_i}, \max\left(0.9 d_s, 0.72 t_{wex}\right)\right) \qquad 0.9 \cdot d_s = 8.72$$

$$d_v = 9.35 \ \text{in} \qquad 0.72 \cdot t_{wex} = 8.64$$

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**.

Therefore the usable capacity is:

$\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_{r1w} := \phi_v \cdot \left(0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r1w} = 11} \ \text{kips}$$

but $\leq$ $V_{r2w} := \phi_v \cdot \left(0.25 \cdot f'_c \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r2w} = 83}$ kips

$$V_{rw} := \min\left(V_{r1w}, V_{r2w}\right) \qquad \boxed{V_{rw} = 11} \ \text{kips}$$

$$V_u := Vstr1_{XW} \qquad \boxed{V_u = 8.7} \ \text{kips}$$

Check that the provided shear capacity is adequate:

Is $V_u = 8.7$ kip $\leq$ $V_{rw} = 11.3$ kip ? $\boxed{check = \text{"OK"}}$

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37.1 Structure Selection

Most pedestrian bridges are located in urban areas and carry pedestrian and/or bicycle traffic over divided highways, expressways and freeway systems. The structure type selected is made on the basis of aesthetics and economic considerations. A wide variety of structure types are available and each type is defined by the superstructure used. Some of the more common types are as follows:

- Concrete Slab
- Prestressed Concrete Girder
- Steel Girder
- Prefabricated Truss

Several pedestrian bridges are a combination of two structure types such as a concrete slab approach span and steel girder center spans. One of the more unique pedestrian structures in Wisconsin is a cable stayed bridge. This structure was built in 1970 over USH 41 in Menomonee Falls. It is the first known cable stayed bridge constructed in the United States. Generally, pedestrian bridges provide the designer the opportunity to employ long spans and medium depth sections to achieve a graceful structure.

Pedestrian boardwalks will not be considered “bridges” when their clear spans are less than or equal to 20 feet, and their height above ground and/or water is less than 10 feet. Boardwalks falling under these constraints will not be required to follow the design requirements in the WisDOT Bridge Manual, but will need to follow the standards established in the *Wisconsin Bicycle Facility Design Handbook.*

37.2 Specifications and Standards

The designer shall refer to the following related specifications:

- "*AASHTO LRFD Bridge Design Specifications*"
- *"AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges",* hereafter referred to as the *"Pedestrian Bridge Guide"*
- See *Standardized Special Provision (STSP)* titled "*Prefabricated Steel Truss Pedestrian Bridge LRFD*" for the requirements for this bridge type

For additional design information, refer to the appropriate Wisconsin Bridge Manual chapters relative to the structure type selected.

The pedestrian live load (PL) shall be as follows: (from "*Pedestrian Bridge Guide")*

- 90 psf **[Article 3.1]**
- Dynamic load allowance is not applied to pedestrian live loads **[Article 3.1]**

The vehicle live load shall be applied as follows: (from *"Pedestrian Bridge Guide")*

- Design for an occasional single maintenance vehicle live load (LL) **[Article 3.2]**

Clear Bridge Width (w)	Maintenance Vehicle
7 ft $\leq$ w $\leq$ 10 ft	H5 Truck (10,000 lbs)
w > 10 ft	H10 Truck (20,000 lbs)

- Clear bridge widths of less than 7 feet need not be designed for maintenance vehicles. **[Article 3.2]**
- The maintenance vehicle live load shall not be placed in combination with the pedestrian live load. **[Article 3.2]**
- Dynamic load allowance is not applied to the maintenance vehicle. **[Article 3.2]**
- Strength I Limit State shall be used for the maintenance vehicle loading. **[Article 3.2, 3.7]**

On Federal Aid Structures FHWA requests a limiting gradient of 8.33 percent (1:12) on ramps for pedestrian facilities to accommodate the physically handicapped and elderly as recommended by the "*American Standard Specifications for Making Buildings and Other Facilities Accessible to, and Usable by, the Physically Handicapped*". This is slightly flatter than the gradient guidelines set by AASHTO which states gradients on ramps should not be more than 15 percent and preferably not steeper than 10 percent.

The minimum inside clear width of a pedestrian bridge on a pedestrian accessible route is 8 feet. (*AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, 2004).*

The width required is based on the type, volume, and direction of pedestrian and/or bicycle traffic.

The vertical clearance on the pedestrian bridge shall be a minimum of 10 feet for bicyclists' comfort and to allow access for maintenance and emergency vehicles. The Wisconsin Department of Natural Resources recommends a vertical clearance on the bridge of at least 12 feet to accommodate maintenance and snow grooming equipment on state trails. Before beginning the design of the structure, the Department of Natural Resources and the Bureau of Structures should be contacted for the vertical clearance requirements for all vehicles that require access to the bridge.

In addition, ramps should have rest areas or landings 5 feet to 6 feet in length which are level and safe. Rest area landings are mandatory when the ramp gradient exceeds 5 percent. Recommendations are that landings be spaced at 60 foot maximum intervals, as well as wherever a ramp turns. This value is based on a maximum gradient of 8.33 percent on pedestrian ramps, and placing a landing at every 5 feet change in vertical elevation. Also, ramps are required to have handrails on both sides. See Standard Details for handrail location and details.

Minimum vertical clearance for a pedestrian overpass can be found in the *Facilities Development Manual (FDM)* Procedure 11-35-1, Attachment 1.8 and 1.9. Horizontal clearance is provided in accordance with the requirement found in *(FDM)* Procedure 11-35-1, Attachment 1.5 and 1.6.

Live load deflection limits shall be in accordance with the provisions of **LRFD [2.5.2.6.2]** for the appropriate structure type.

Pedestrian loads, as described in the AASHTO LRFD Bridge Design Specifications, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

37.3 Protective Screening

Protective Screening is recommended on all pedestrian overpasses due to the increased number of incidents where objects were dropped or thrown onto vehicles traveling below. Several types of screening material are available such as aluminum, fiberglass and plastic sheeting, and chain link type fencing. A study of the various types of protective screening available indicates that chain link fencing is the most economical and practical for pedestrian overpasses. For recommended applications refer to the Standard Details.

The top of the protective screening may be enclosed (not required) with a circular section in order to prevent objects from being thrown over the sides and to discourage people from climbing on (over) the top. The opening at the bottom is held at a 1 inch clearance to prevent objects from being pushed under the fence.

The core wire of the fence fabric shall be a minimum of 9 gauge (0.148 inch) thickness, galvanized and woven in a 2 inch mesh. A 1 inch mesh may be used in highly vulnerable areas. A vinyl coating may also be used for aesthetic purposes. Add a special provision to the contract if these additional features are used. Special provisions for common items are available as STSP’s or on the Wisconsin Bridge Manual website.

Region project staff should be consulted with regards to fencing preferences.

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Table of Contents

38.1 Introduction

The principles of designing railroad structures are similar to those for structures carrying highways. However, structures carrying railways have much heavier loadings than those subject to highway loadings due to increased dead load, live load and impact required for railways.

The general features of design, loadings, allowable stresses, etc., for railway structures are controlled by the specifications of the American Railway Engineering and Maintenance-of-Way Association (AREMA). The different railroad companies vary somewhat in their interpretation and application of these specifications as stated in the *AREMA Manual for Railway Engineering* (hereafter referred to as *AREMA Manual).* Requirements for railroad structures vary with the railroad company whose tracks are carried by the structure, and are sometimes varied by the same company in different locations. The *AREMA Manual* provides for design of railroad structures using Allowable Stress Design (ASD) and Load Factor Design (LFD) methods. The Load and Resistance Factor Design (LRFD) method is currently not used. Designers should bear in mind that specifications were developed for more or less typical conditions. If a structure is unusual in some respects, designers should use their best engineering judgment in selection of proper design criteria. Most railroad companies permit and prefer high strength bolted or shop welded steel plate girders, reinforced concrete or prestressed concrete members in bridge construction.

Safety of the traveler on the highway under the structure and uniformity of track surface dictates that the full ballast section of the railway be carried on the structure. The relatively heavy loadings and high impact factor together with the span and clearance requirements usually found in underpass structures, practically limit the choice of materials for the superstructure to structural steel. The floor under the ballast may be steel plate or reinforced concrete and the substructures could be reinforced concrete or structural steel as conditions warrant.

The *AREMA Manual* covers all phases of railway design, construction, maintenance and operation. It is divided into sections and chapters. Chapter 8, Concrete Structures and Foundations, governs the design and construction of plain and reinforced concrete members, rigid concrete structures, retaining walls, pile foundations, substructures of railway structures, etc. Chapter 15 - Steel Structures, governs the design and construction of steel railroad structures.

In this chapter, reference will be made to specific articles of the *AREMA Manual* as required.

The AREMA specifications are revised annually and it is essential that the latest revisions be used. The *AREMA Manual* is a guideline only and should be followed as a starting point in design.

Railroad companies are essentially conservative as their primary interest is the safety of their trains and human lives. Their requirements are usually based upon their operating experience and are set up with that interest in view.

38.2 Design Specifications and Design Aids

38.2.1 Specifications

Allowable stresses are provided in different chapters and sections of the *AREMA Manual*.

Refer to the design, construction, maintenance and operation related materials as presented in the stated sections of the following specifications:

American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering

- Chapter 8 - Concrete Structures and Foundations
- Chapter 15 - Steel Structures – (Design, Fabrication and Construction)
- Chapter 28 – Clearances - (Horizontal and Vertical)

AASHTO Standard Specifications for Highway Bridges, 17th Edition

Wisconsin Standard Specifications for Highway and Structure Construction

38.2.2 Design Aids

In the design of railroad structures the only short cut available is a method of computing Live Load Moments, Shears and Reactions by the use of tables which can be found in Section 1.15 of Chapter 15, part 1 of the *AREMA Manual*. This table reflects Cooper E80 Live Loading shown in Figure 38.3-5. All the moment, shear and reaction values are for one rail (one-half track load) only and all the values can be prorated (directly proportional) for smaller or larger Cooper's E live loadings.

Floor beam spacings in through plate-girder railroad structures may be determined by a number of things, but consideration should be given to the transverse stiffener spacings of the girders. It is very convenient to have the floor beam spacing in multiples of stiffener spacings.

For ballasted structures, all lateral forces will be carried by the steel ballast plate which is extremely rigid and lateral bracing will not be required.

38.2.3 Horizontally Curved Structures

The latest AREMA specifications as well as individual railroad company's interpretation and application of the *AREMA Manual* should be followed in designing and detailing curved structures. There is considerable information available on designing curved steel girders. Most of the methods require computer programs that may be difficult to use. The Approximate Method of Design developed by USS Corporation is an accepted approach for horizontally curved girders.

38.2.4 Railroad Approval of Plans

There is a need to get the individual railroad company's unique design requirements. Smaller companies such as Wisconsin & Southern may rely on AREMA requirements and DOT experience.

Prior to starting the preliminary design, the Bureau of Structures (BOS) should receive the railroad company's current standards and design policy guidelines.

Before the preliminary plan is prepared, the Regional Project Manager, BOS and Bureau of Rails and Harbors (BRH) should review the particular railroad company's design standards for compliance with 23 CFR and DOT policy, and for compatibility and practicality with unique project features.

The preliminary structure plan should be prepared and submitted to the railroad company for approval after the above steps have been completed.

Detailed structure design should not begin until the railroad company has approved the preliminary plan.

The bridge designer should work directly with the railroad's bridge engineering office where interpretation of requirements or clarification of design details is needed.

The final structure plan and special provisions need to be sent to and approved by the railroad company before the project is authorized for letting.

38.3 Design Considerations

38.3.1 Superstructure

38.3.1.1 Methods of Design, Selection Type and Superstructure General

The preferred types of railroad structures are as follows:

- Rolled or welded girders for spans of 50 feet or less
- Bolted or welded plate girders for spans over 50 to 150 feet
- Bolted or welded trusses for spans over 150 feet

The superstructures of grade separations carrying railroad traffic are usually of beam and girder construction. The spans are generally too short for economical use of trusses and other factors, such as appearance, maintenance, etc., discourage their use.

Floor systems in beam and girder construction, for moderate spans, may be divided into two general classes:

- One-way Floor System
- Two-way Floor System

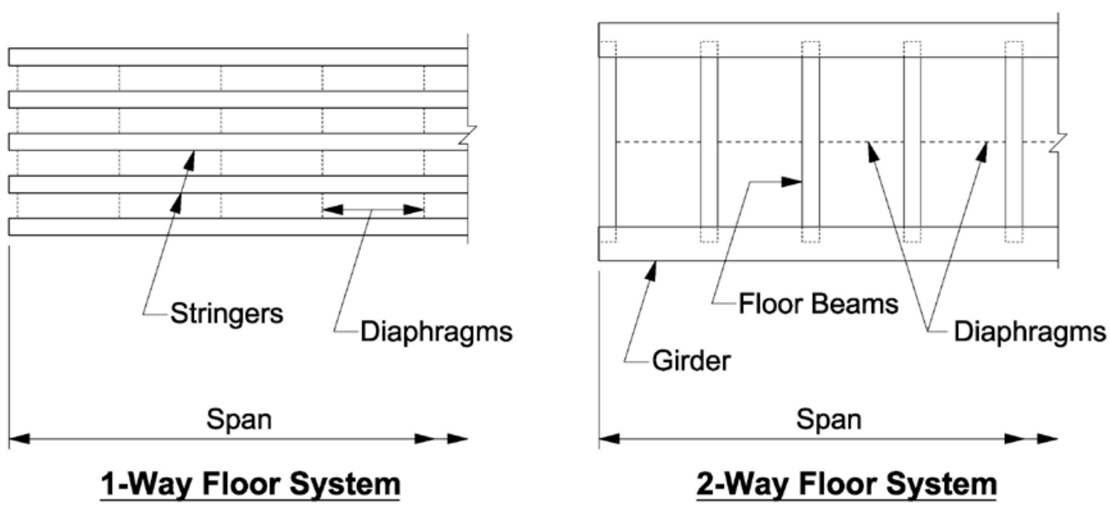

Figure 38.3-1
Types of Floor Systems

The One-way floor system is always a deck structure and is particularly adaptable for structures carrying several tracks or subject to future widening or other controls which make a deck structure desirable. The Two-way floor system may be either a through plate-girder or deck structure depending upon whether the floor beams are placed near the bottom or the top flange of the girders. It is usually desirable to keep the depth of structure (base of rail to low steel) at a minimum. Therefore, most underpasses with two-way floor systems are through structures as shown in Figure 38.3-2.

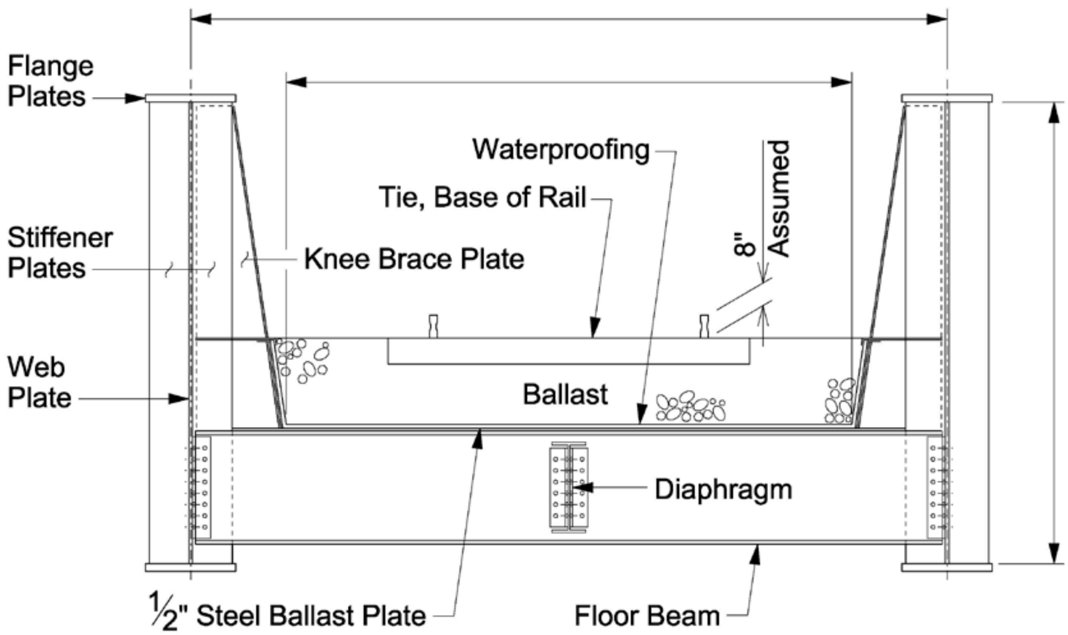

Figure 38.3-2
Typical Section of Through-Girder Bridge
(Two-Way Floor System)

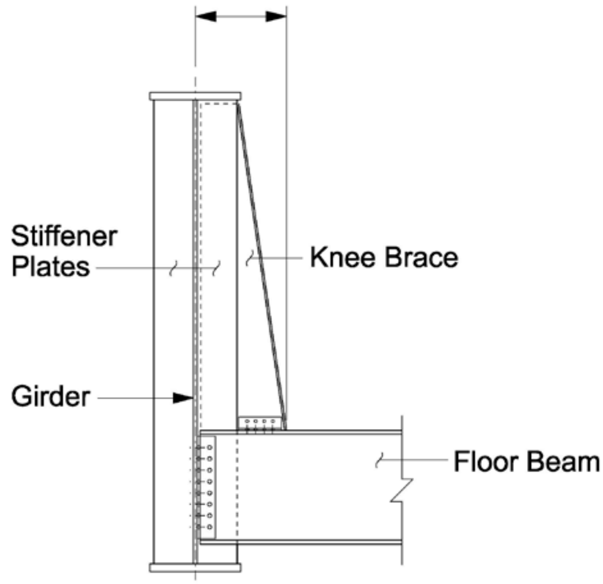

Figure 38.3-3
Knee Brace for Through-Girder Bridge

Through girders should be laterally braced with gusset plates or knee braces with solid webs connected to the stiffeners as shown in Figure 38.3-3. The *AREMA Manual* limits the spacing of knee braces to 12 feet maximum. They also dictate that the type of braces are to be web plates with flanges. Since knee braces support the top flanges against buckling, smaller values of L/b (L = unsupported distance between the nearest lines of fasteners or welds, or between the roots of rolled flanges)/ (b= flange width) produce higher allowable fiber stresses in the top flanges.

Almost all railroad structures are usually simple spans for the following reason:

Usually the maximum negative moment over the support is nearly equal to the positive moment of the simple beam. In some combinations, the continuous beam negative moments may be greater than the simple beam positive moment because of the unfavorable Live Load placement in the spans. Continuity introduces complications and it is questionable if any real saving is realized by its use.

In railroad structures, spacing of the through girders is governed by AREMA specifications for Steel Railway Structures. The spacing should be at least 1/20 of the span or should be adequate to insure that the girders and other structural components provide required clearances for trains, whichever is greater. The requirement of lateral clearance each side of track centerline for curved alignment should be as per latest AREMA specifications. A typical girder inside elevation view is shown in Figure 38.3-4.

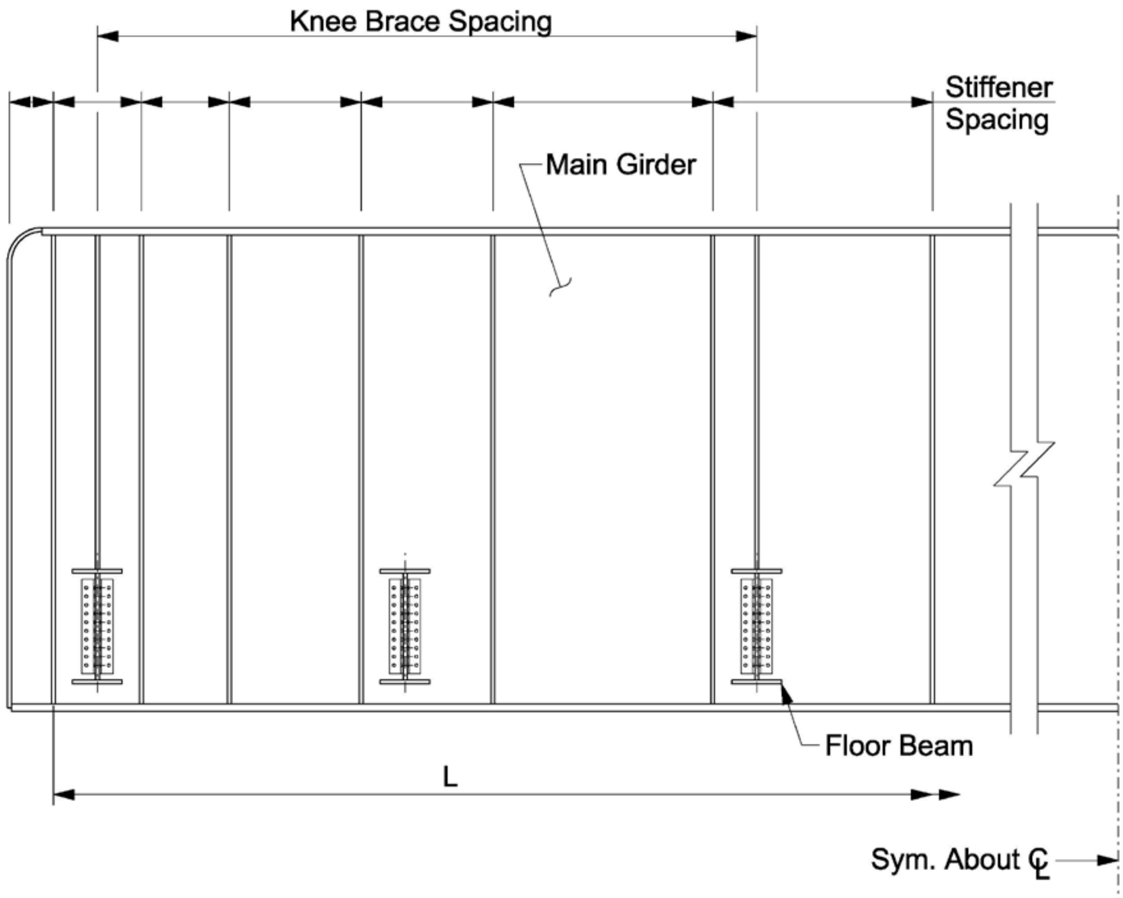

Figure 38.3-4
Typical Through-Girder Inside Elevation

38.3.1.2 Ballast Floor

The superstructure includes the ballast floor, girders and girder bearings to the top of the masonry. The thickness of the ballast floor shall not be less than ½ inches for steel plate or 6 inches for reinforced or prestressed concrete. For concrete floor, thickness is measured from top of bars or cover plate and the reinforcement is usually #4 bars at 6 inches both at top and bottom.

38.3.1.3 Dead Load

Dead Load consists of weight of track rails and fastenings, ballast and ties, weight of waterproofing, ballast plate, floor beams, etc. Most of the load carried by each girder is transmitted to it by the floor beams as concentrated loads. Computations are simpler, however, if the floor beam spacings are ignored and the girder is treated as if it received load from the ballast plate. Moments and shears computed with this assumption are sufficiently accurate for design purposes because of the relatively close spacing of the floor beams. Thus, the dead load on the girder may be considered uniformly distributed.

38.3.1.4 Live Load

The *AREMA Manual* recommends that design be based on Cooper E80 Live Loading as shown in Figure 38.3-5. Heavier Cooper E loadings will result in directly proportional increases in the concentrated and uniform live loadings shown in Figure 38.3-5.

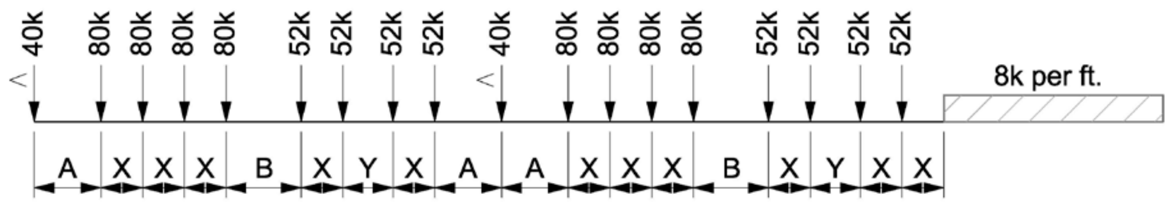

Figure 38.3-5
Cooper E80 Live Loading

X = 5 ft A = 8 ft
Y = 6 ft B = 9 ft

To account for the effect of multiple tracks on a structure, the portions of full live load on the tracks may be taken as:

Number of Tracks	Loading
Two tracks	Full live load
Three tracks	Full live load on two tracks, one-half full live load on the third track
Four tracks	Full live load on two tracks, one-half on one track, and one quarter on the remaining track
More than four tracks	As specified by the Engineer

Table 38.3-1
Live Load vs. Number of Tracks

The selection of the tracks for these loads shall be such as will produce the greatest live load stress in the member.

38.3.1.5 Live Load Distribution

On open-deck structures, ties within a length of 4 feet but not more than three ties may be assumed to support a wheel load. The live load should be considered a series of concentrated loads, however, for the design of beams and girders. No longitudinal distribution of wheel loads shall be assumed.

When two or more longitudinal beams per rail are properly diaphragmed in accordance with *AREMA Manual* Chapter 15, and symmetrically spaced under the rail, they shall be considered as equally loaded.

For ballasted-deck structures, live load distribution is based on the assumption of standard cross ties at least 8 feet long, about 8 inches wide, and spaced not more than 2 feet on centers, with at least 6 inches of ballast under the ties. For deck design, each axle load should be uniformly distributed over a length of 3 feet plus the minimum distance from bottom of tie to top of beams or girders, but not more than 5 feet or the minimum axle spacing of the loading. In the lateral direction, the axle load should be uniformly distributed over a width equal to the length of tie plus the minimum distance from bottom of tie to top of beams or girders.

Transverse steel beams without stringers

For ballasted concrete decks supported by transverse steel beams without stringers, the portion of the maximum axle load to be carried by each beam is given by:

$$P = \frac{1.15\,AD}{S}$$

Where:

P = Load on a beam from one track

A = Axle Load

S = Axle spacing (ft)

D = Effective beam spacing (ft)

For bending moment, within the limitation that D may not exceed either axle or beam spacing, the effective beam spacing may be computed from:

$$D = d\left(\frac{1}{1+\frac{d}{aH}}\right)\left(0.4+\frac{1}{d}+\frac{\sqrt{H}}{12}\right)$$

Where:

a = Beam span (ft)

H = $\frac{nI_b}{ah^3}$

n = Ratio of modulus of elasticity of steel to that of concrete

I_b = Moment of inertia of beam (in^4)

h = Thickness of concrete deck (in)

d = Beam spacing (ft)

For end shear, D = d

The load P shall be applied as two equal concentrated loads on each beam at each rail, equal to P/2. Lateral distribution of such loads shall not be assumed.

The value for “D” should be taken equal to “d” for structures without a concrete deck or for structures where the concrete slab extends over less than 75% of the floor beam.

Where “d” exceeds S, P should be the maximum reaction of the axle loads with the deck between beams acting as a simple span.

For longitudinal steel beams or girders

For ballasted decks supported on longitudinal girders, axle loads should be distributed equally to all girders whose centroids lie within a lateral width equal to length of tie plus twice the minimum distance from bottom of tie to top of girders. Distribution of loads for other conditions shall be determined by a recognized method of analysis.

38.3.1.6 Stability

For spans and towers, stability should be investigated with live load on only one track, the leeward one for structures with more than one track. The live load should be 1200 plf, without impact.

38.3.1.7 Live Load Impact

AREMA Manual Chapter 15 specifies the impact forces to be used and how they are to be applied. Impact forces should be applied vertically and equally at top of each rail. Impact, I, expressed as a percentage of axle loads, is given for open-deck structures by the following equations and modified by a factor determined by the number of tracks to be supported. For ballasted deck structures the percentage to be used shall be 90% of that specified for open deck structures.

For rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.)

For L less than 80 feet

$$I = RE + 40 - \frac{3L^2}{1600}$$

For L = 80 feet or more

$$I = RE + 16 + \frac{600}{L - 30}$$

For steam locomotives with hammer blow:

For beam spans, stringers, girders, floorbeams, posts of deck truss spans carrying load from floorbeams only, and floorbeam hangers:

For L less than 100 feet

$$I = RE + 60 - \frac{L^2}{500}$$

For L = 100 feet or more

$$I = RE + 10 + \frac{1800}{L - 40}$$

For truss spans

$$I = RE + 15 + \frac{4000}{L + 25}$$

Where:

RE	=	Either 10% of axle load or 20% of the wheel load.
L	=	Length in feet, center to center of supports for stringers, transverse floorbeams without stringers, longitudinal girders and trusses (main members), or length in feet, of the longer adjacent supported stringers, longitudinal beam, girder or truss for impact in floor beams, floor beam hangers, subdiagonals of trusses, transverse girders and viaduct columns.

For members receiving load from more than one track, the impact percentage shall be applied to the live load on the number of tracks designated below.

Load received from two tracks	
For L less than 175 ft	Full impact on two tracks
For L from 175 to 225 ft	Full impact on one track and a percentage of full impact on the other as given by the formula, 450 - 2L
For L greater than 225 ft	Full impact on one track and none on the other
Load received from more than two tracks	
For all values of L	Full impact on any two tracks

Table 38.3-2
Live Load Impact

38.3.1.8 Centrifugal Forces on Railroad Structures

On curves, a centrifugal force corresponding to each axle load should be applied horizontally through a point 6 feet above the top of rail. This distance should be measured in a vertical plane along a line that is perpendicular to and at the midpoint of a radial line joining the tops of the rails. This force should be taken as a percentage, C, of the specified axle load without impact.

$$C = 0.00117S^2D$$

Where:

S = Speed (mph)

D = Degree of curve = 5729.65/R

R = Radius of curve (ft)

Preferably, the section of the stringer, girder or truss on the high side of the superelevated track should be used also for the member on the low side, if the required section of the low-side member is smaller than that of the high-side member.

If the member on the low side is computed for the live load acting through the point of application defined above, impact forces need not be increased. Impact forces may, however, be applied at a value consistent with the selected speed in which case the relief from centrifugal force acting at this speed should also be taken into account.

38.3.1.9 Lateral Forces From Equipment

For bracing systems or for longitudinal members entirely without a bracing system, the lateral force to provide for the effect of the nosing of equipment, such as locomotives, (in addition to the other lateral forces specified) should be a single moving force equal to 25% of the heaviest axle load. It should be applied at top of rail. This force may act in either lateral direction at any point of the span. On spans supporting multiple tracks, the lateral force from only one track should be used.

The resulting stresses to be considered are axial stresses in members bracing the flanges of stringer, beam and girder spans, axial stresses in the chords of truss spans and in members of cross frames of such spans, and stresses from lateral bending of flanges of longitudinal flexural members having no bracing system. The effects of lateral bending between braced points of flanges, axial forces in flanges, vertical forces and forces transmitted to bearings shall be disregarded.

38.3.1.10 Longitudinal Forces on Railroad Structures

The longitudinal force from trains should be taken as 15% of the live load without impact.

Where the rails are continuous (either welded or bolted joints) across the entire structure from embankment to embankment, the effective longitudinal load shall be taken as L/1200 (where L is the length of the structure in feet) times the load specified above (15% of live load), but the value of L/1200 used shall not exceed 0.80.

Where rails are not continuous, but are interrupted by a moveable span, sliding rail expansion joints or other devices, across the entire structure from embankment to embankment, the effective longitudinal force should be taken as 15% of live load.

The effective longitudinal force should be taken on one track only. It should be distributed to the various components of the supporting structure, taking into account their relative stiffnesses, where appropriate, and the type of bearings.

The effective longitudinal force should be assumed to be applied at base of rail.

38.3.1.11 Wind Loading on Railroad Structures

AREMA Manual Chapter 15 provides the details of wind loading on railroad structures.

The wind load shall be considered as a moving load acting in any horizontal direction. On the train it shall be taken at 300 plf on the one track, applied 8 feet above the top of rail. On the structure it shall be taken at 30 psf on the following surfaces:

- For girder spans, 1.5 times the vertical projection of the span.
- For truss spans, the vertical projection of the span plus any portion of the leeward trusses not shielded by the floor system.
- For viaduct towers and bents, the vertical projections of all columns and tower bracing.

The wind load on girder spans and truss spans, however, shall not be taken at less than 200 plf for the loaded chord or flange, and 150 plf for the unloaded chord or flange.

The wind load on the unloaded structure shall be assumed at 50 psf of surface as defined in the bulleted items above.

38.3.1.12 Loads from Continuous Welded Rails

Section 8.3 of Chapter 15 *AREMA Manual* describes the details of the effect of continuous welded rails. Forces in continuous welded rail may be computed from the following equations:

$$I.F. = 38WT$$

$$R.F. = \frac{WDT}{150}$$

Where:

I.F.	=	Internal force in two rails (lb); compression for temperature rise, tension for temperature fall.
R.F.	=	Radial force in two rails, (lb/ft of bridge); acting toward outside of curve for temperature rise, toward inside for temperature fall.
W	=	Weight of one rail (lb/yd)
T	=	Temperature change (°F)
D	=	Degree of curvature

38.3.1.13 Fatigue Stresses on Structures

The major factors governing fatigue strength are the number of stress cycles, the magnitude of the stress range, and the type and location of constructional detail. The number of stress cycles, N, to be considered shall be selected from Table 15-1-7 of Chapter 15 *AREMA Manual*, unless traffic surveys or other considerations indicate otherwise. The selection depends on the span length in the case of longitudinal members, and on the number of tracks in the case of floor beams and hangers.

Formulas for allowable fatigue stresses on structures recommended by AREMA are dependent primarily on the strength of the material, the stress range, number of stress cycles and a stress ratio R.

The stress range, S_R, is defined as the algebraic difference between the maximum and minimum calculated stress due to dead load, live load, impact load and centrifugal force. If live load, impact load and centrifugal force result in compressive stresses and the dead load stress is compression, fatigue need not be considered.

The type and location of the various constructional details are categorized in Table 15-1-9 and illustrated in Figure 15-1-5 *AREMA Manual*. The stress range for other than Fracture Critical Members shall not exceed the allowable fatigue stress range, $S_R$fat, listed in Table 15-1-10.

The stress range for Fracture Critical Members shall not exceed the allowable fatigue stress range $S_R$fat, listed in Table 15-1-10 (see Note 2) *AREMA Manual*.

38.3.1.14 Live Load Deflection

The deflection of the structure shall be computed for the live loading plus impact loading condition producing the maximum bending moment at mid-span for simple spans. In this computation, gross moment of inertia shall be used for flexural members and gross area of members for trusses. For members with perforated cover plates, the effective area shall be used.

The structure shall be so designed that the computed deflection shall not exceed 1/640 of the span length, center to center of bearings for simple spans.

38.3.1.15 Loading Combinations on Railroad Structures

Every component of superstructure and substructure should be proportioned to resist all combinations of forces applicable to the type of structure and its site. Members subject to stresses resulting from dead load, live load, impact load and centrifugal force shall be designed so that the maximum stresses do not exceed the basic allowable stresses of Section 1.4, and the stress range does not exceed the allowable fatigue stress range allowed by AREMA specifications.

The basic allowable stresses of Section 1.4 shall be used in the proportioning of members subject to stresses resulting from wind loads only, as specified in *AREMA Manual*, Article 1.3.8.

With the exception of floorbeam hangers, members subject to stresses from other lateral or longitudinal forces, as well as to the dead load, live load, impact and centrifugal forces may be proportioned for 125% of the basic allowable unit stresses, without regard for fatigue. But the section should not be smaller than required with basic unit stresses or allowable fatigue stresses when those lateral or longitudinal forces are not present.

Increase in allowable stress permitted by the previous paragraph shall not be applied to allowable stress in high strength bolts.

38.3.1.16 Basic Allowable Stresses for Structures

Design of steel railroad structures usually is based on a working stress level that is some fraction of the minimum yield strength of the material. This value commonly is 0.55, allowing a safety factor of 1.82 against yield of the steel. The basic allowable stresses for structural steel, rivets, bolts and pins to be used in proportioning the parts of a structure are furnished in Table 15-1-11 in the *AREMA Manual* Chapter 15.

38.3.1.17 Length of Cover Plates and Moment Diagram

The dead load moment diagram is a parabola with mid-ordinate showing the maximum dead load moment. Determination of the exact shape of the envelope for the live load moment involves long and tedious calculations. The procedure consists of dividing the span into parts and finding the maximum moment at each section. The smaller the divisions, the more accurate the shape of the curve and the more involved and tedious the calculations.

Fortunately, a parabola with mid-ordinate equal to the tabular value for maximum moment, Section 1.15, *AREMA Manual* Chapter 15, very nearly encloses the envelope. Therefore the shape of the moment diagram of DL + LL + I is parabolic for all practical purposes. Knowing the maximum ordinate, the designer can compute the other values and draw the moment curve.

The resisting moment diagram can be superimposed upon actual moment diagram described above. The theoretical end of cover plates can be determined from these moment envelopes.

The AREMA specifications require that flange plates shall extend far enough to develop the capacity of the plate beyond the theoretical end. This method of determining the theoretical end of cover plates, on girders proportioned for deflection is not exact, but is acceptable for design purposes.

38.3.1.18 Charpy V-Notch Impact Requirements

Recognizing the need to prevent brittle fracture failures of main load carrying structural components, AREMA specifications have provisions for Charpy V-Notch impact testing and the values for steel other than fracture critical members are tabulated in Table 15-1-2 in *AREMA Manual*.

The design requirements for materials of Fracture Critical Members shall further comply with the Fracture Control Plan specified in *AREMA Manual* Chapter 15, Section 1.14. The Engineer shall designate on the plans which members or member components fall in the category of Fracture Critical Members.

38.3.1.19 Fracture Control Plan for Fracture Critical Members

For purposes of the Fracture Control Plan, Fracture Critical Members or member components (FCM's) are defined as those tension members or tension components of members whose failure would be expected to result in collapse of the structure or inability of the structure to perform its design function.

AREMA specifications have elaborate descriptions of the Fracture Control Plan which has special requirements for the materials, fabrication, welding, inspection and testing of Fracture Critical Members and member components in steel railway structures. The provisions of this plan are to:

- Assign responsibility for designating which steel railway structure members or member components, if any, fall in the category of "Fracture Critical".

- Require that fabrication of FCM or member components be done in plants having personnel, organization, experience, procedures, knowledge and equipment capable of producing quality workmanship.

- Require that all welding inspectors demonstrate their competency to assure that welds in FCM or member components are in compliance with this plan.

- Require that all non-destructive testing personnel demonstrate their competency to assure that tested elements of FCM or member components are in compliance with this plan.

- Specify material toughness values for FCM or member components.

- Supplement recommendations for welding contained elsewhere in AREMA specifications.

Charpy V-Notch (CVN) impact test requirements for steels in FCM's shall be always followed as given in *AREMA Manual* Table 15-1-14. Impact tests shall be in accordance with the CVN tests as governed by ASTM Designation A673 for frequency of testing P (impact). Impact tests shall be required on a set of specimens taken from each end of each plate. Wisconsin currently specifies its steel to Zone 3 when impacts are required on railroad structures. Since Wisconsin Standard Specifications say Zone 2, Zone 3 must be stated on the plans.

38.3.1.20 Waterproofing Railroad Structures

AREMA specifications on waterproofing railroad structures apply to materials and construction methods for an impervious membrane and auxiliary components to protect structures from harmful effects of water. Railroad structures which require waterproofing shall be designed so that they can be waterproofed by the methods and with the materials specified in AREMA specifications. The materials for waterproofing and the methods of application should be such as to insure that the waterproofing will be retained by bond, anchorage or other adequate means, in its original position as applied to the surface to be waterproofed.

The membrane shall consist of one of the following types, as described below.

- Minimum 3/32 inch thick butyl rubber sheeting secured with an approved adhesive.

- Heavy Duty Bituthene or Protecto Wrap M400 may be used.

- Rubberized asphalt with plastic film or 4 feet x 8 feet sheets of preformed board membrane with maximum thickness of ½ inch.

The butyl rubber sheeting, rubber membrane splicing cement and the butyl gum splicing tape shall be in accordance with the requirements for membrane waterproofing as specified in part 29 of Chapter 8 of the *AREMA Manual*. Cement for splicing rubber membrane shall be a self-vulcanizing butyl rubber compound and shall be applied at a minimum rate of 2 gallons/100 square feet.

38.3.2 Substructure

38.3.2.1 Abutments and Retaining Walls

The abutments for railroad structures are essentially bearing walls subject to lateral pressure. The design procedure is similar to that required for a retaining wall. The typical section is shown in Figure 38.3-6.

AREMA Manual Chapter 8, Part 5, governs the requirements for retaining walls. They are essentially the same as AASHTO requirements providing the backfill is of sandy material.

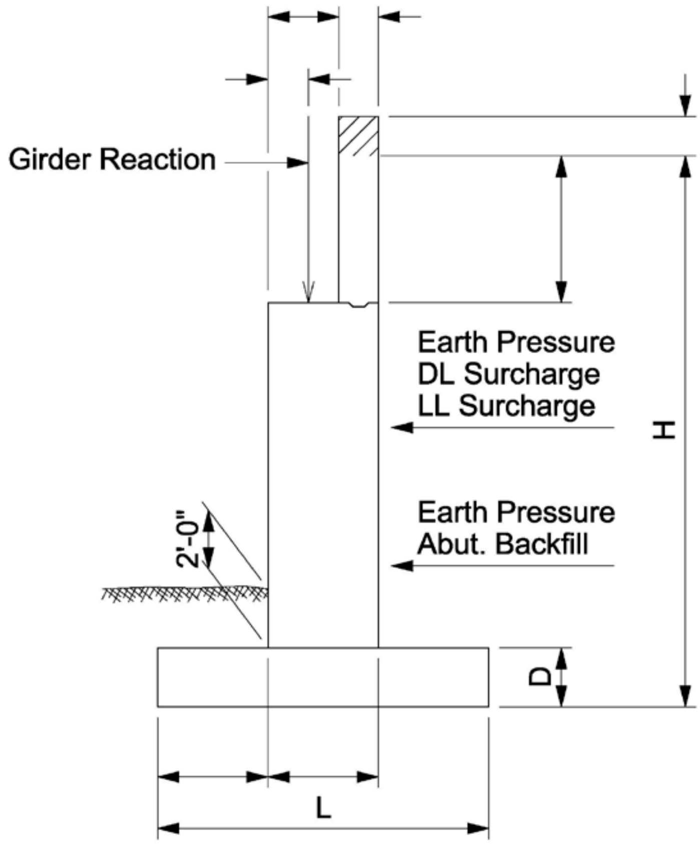

Figure 38.3-6
Typical Abutment

1. Field Survey

   Sufficient information shall be furnished, in the form of a profile and cross sections or a topographic map, to determine the structural requirements. Present grades and alignments of tracks and roads shall be indicated, together with the records of high water, low water and depth of scour, the location of underground utilities, and information concerning any structures that may affect or be affected by the construction.

2. Subsurface exploration

   Specifications provided by *AREMA Manual* Chapter 8 and Part 22 should be followed.

3. Character of backfill

   Backfill is defined as all material behind the wall, whether undisturbed ground or fill, that contributes to the pressure against the wall.

   AREMA Manual Chapter 8, Table 8-5-1 classifies the type of backfill materials for retaining walls.

Type	Description
Type 1	Coarse-grained soil without admixtures of fine soil particles, very free draining (clean sand, gravel or broken stone)
Type 2	Coarse-grained, soil of low permeability due to admixtures of particles of silt size
Type 3	Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones
Type 4	Soft or very soft clay; organic silt; or soft silty clay
Type 5	Medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains

Table 38.3-3
Classification of Backfill Material

4. Computation of Earth Pressure

   Values of the unit weight, cohesion and angle of internal friction of the backfill material shall be determined directly by means of soil tests or, if the expense of such tests is not justifiable, refer to Table 8-5-2 in *AREMA Manual* for the soil types defined above. Unless the minimum cohesive strength of backfill material can be evaluated reliably the cohesion shall be neglected and only the internal friction considered.

   When the backfill is assumed to be cohesionless; when the surface of the backfill is or can be assumed to be plane; when there is no surcharge load on the surface of the backfill; or when the surcharge can be converted into an equivalent uniform earth surcharge, Rankine's or Coulomb's formulas may be used under the conditions to which each applies. Formulas and charts given in *AREMA Manual* Chapter 8, Part 5 Commentary and the trial wedge method also presented in this Commentary are both applicable.

5. Computation of Loads Exclusive of Earth Pressure

In the analysis of retaining walls and abutments, due account shall be taken of all superimposed loads carried directly on them, such as building walls, columns, or bridge structures; and of all loads from surcharges caused by railroad tracks, highways, building foundations or other loads supported on the fill behind the walls.

In calculating the surcharge due to track loading, the entire load shall be taken as distributed uniformly over a width equal to the length of the tie. Impact shall not be considered unless the bearings are supported by a structural beam, as in a spill-through abutment.

6. Stability Computation

The resultant force on the base of a wall or abutment shall fall within the middle third if the structure is founded on soil, and within the middle half if founded on rock, masonry or piles. The resultant force on any horizontal section above the base of a solid gravity wall should intersect this section within its middle half. If these requirements are satisfied, safety against overturning need not be investigated.

The factor of safety against sliding at the base of the structure is defined as the sum of the forces at or above base level available to resist horizontal movement of the structure divided by the sum of the forces at or above the same level tending to produce horizontal movement. The numerical value of this factor of safety shall be at least 1.5. If the factor of safety is inadequate, it shall be increased by increasing the width of the base, by the use of a key, by sloping the base upward from heel to toe or by the use of battered piles.

In computing the resistance against sliding, the passive earth pressure of the soil in contact with the face of the wall shall be neglected. The frictional resistance between the wall and a non-cohesive subsoil may be taken as the normal pressure on the base times the coefficient of friction of masonry on soil. For coarse-grained soil without silt, the coefficient of friction may be taken as 0.55; for coarse-grained soil with silt, as 0.45; and for silt as 0.35. For concrete on sound rock the coefficient of friction may be taken as 0.60.

The factor of safety against sliding on other horizontal surfaces below the base shall be investigated and shall not be less than 1.5.

38.3.2.2 Piers

A pier is a structural member of steel, concrete or masonry that supports the vertical loads from the superstructure, as well as the horizontal loads not resisted by the abutments. Also, piers must be capable of resisting forces they may receive directly such as wind loads, floating ice and debris, expanding ice, hydrokinetic pressures and vehicle impact.

The connection between pier and superstructure is usually a fixed or expansion bearing which allows rotation in the longitudinal direction of the superstructure.

The types of piers most frequently used in railroad structures can be classified in one of the following categories:

- Pile Bents
- Solid Single Shaft
- Multi-Column Frames
- Individual Column in Line
- Steel Section

38.3.2.3 Loads on Piers

38.3.2.3.1 Dead Load and Live Loading

Dead load and live load comes from the superstructure on to the pier as girder reactions.

38.3.2.3.2 Longitudinal Force

The longitudinal force from trains shall be taken as 15 percent of the live load without impact. Where the rails are continuous (either welded or bolted joints) across the entire structure from embankment to embankment, the effective longitudinal force shall be taken as L/1200 (where L is the length of the structure in feet) times the force specified above (follow AREMA specifications), but the value of L/1200 shall not exceed 0.80.

The effective longitudinal force shall be assumed to be applied at the top of the supporting structure.

38.3.2.3.3 Stream Flow Pressure

All piers and other portions of structures which are subject to the force of flowing water or drift shall be designed to resist the maximum stresses induced thereby.

38.3.2.3.4 Ice Pressure

The effects of ice pressure, both static and dynamic, shall be accounted for in the design of piers and other portions of the structure where, in the judgment of the engineer, conditions so warrant. The values of effective ice pressure furnished in AASHTO specifications may be used as a guide.

38.3.2.3.5 Buoyancy

Buoyancy shall be considered as it affects the design of the substructure including piling.

38.3.2.3.6 Wind Load on Structure

The wind load acting on the structure shall be assumed as 45 psf on the vertical projection of the structure, applied at the center of gravity of the vertical projection. The wind load shall be assumed to act horizontally, in a direction perpendicular to the centerline of the track.

38.3.2.3.7 Wind Load on Live Load

A moving load of 300 plf on the train shall be applied 8 feet above the top of the rail horizontally in a direction perpendicular to the centerline of the track.

38.3.2.3.8 Centrifugal Force

On curves, a centrifugal force corresponding to each axle load shall be applied horizontally through a point 6 feet above the top of rail measured along a line perpendicular to the line joining the tops of the rails and equidistant from them. This force shall be the percentage of the live load computed from the formulas in 38.3.1.8.

38.3.2.3.9 Rib Shortening, Shrinkage, Temperature and Settlement of Supports

The structure shall be designed to resist the forces caused by rib shortening, shrinkage, temperature rise and/or drop and the anticipated settlement of supports. The following values for range of temperature and coefficient of thermal expansion apply to Wisconsin steel structures.

Temperature range	90°F
Coefficient of thermal expansion	0.0000065/°F

38.3.2.3.10 Loading Combinations

The following groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned for the group of loads that produce the most critical design condition.

Service Load Design

The group loading combinations for Service Load Design are as follows:

Load Case	Load Combinations	Allowable Percentage of Basic Unit Stress
Group I	D + L + I + CF + E + B + SF	100
Group II	D + E + B + SF + W	125
Group III	Group I + 0.5W + WL + LF + F	125
Group IV	Group I + OF	125
Group V	Group II + OF	140
Group VI	Group III + OF	140
Group VII	Group I + ICE	140
Group VIII	Group II + ICE	150
No increase in allowable unit stresses shall be permitted for members or connections carrying wind load only.		

<u>Table 38.3-4</u>
Service Load Design

<u>Load Factor Design</u>

The group loading combinations for Load Factor Design are as follows:

Group I	1.4 (D + 5/3 (L+I) + CF + E + B + SF)
Group IA	1.8 (D + L + I + CF + E + B + SF)
Group II	1.4 (D + E + B + SF + W)
Group III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)
Group IV	1.4 (D + L + I + CF + E + B + SF + OF)
Group V	Group II + 1.4 (OF)
Group VI	Group III + 1.4 (OF)
Group VII	1.0 (D + E + B + EQ)
Group VIII	1.4 (D + L + I + E + B + SF + ICE)
Group IX	1.2 (D + E + B + SF + W + ICE)
The load factors given are only intended for designing structural members by the load factor concept. The actual loads should not be increased by these factors when designing for foundations (soil pressure, pile loads, etc.). The load factors are not intended to be used when checking for foundation stability (safety factors against overturning, sliding, etc.) of a structure.	

<u>Table 38.3-5</u>
Load Factor Design

38.3.2.4 Pier Protection for Overpass Structures

Pier protection should be placed according to the railroad company involved, as they each have different requirements. For minimum requirements, refer to Standard for Highway Over Railroad Design Requirements. Check with the railroad company to determine if they want to extend crash wall beyond columns. Usually they do not.

Crash walls are not required on team tracks and spur tracks as these are for storage or loading and unloading on secondary lines.

Temporary sheet piling may be required by the railroad company during pier and footing construction. All sheet pilings have to be removed after completion of overpass structures. Refer to Standard Highway Over Railroad Design Requirements.

On rehabilitated or widened structures, past practice is to extend the existing protection. If the structure does not have any crashwall protection, past practice is to widen the pier in line with the existing as-built pier provided there is no reduction in horizontal clearance; even though it does not meet current standard clearance criteria.

38.3.2.5 Pier Protection Systems at Spans Over Navigable Streams

38.3.2.5.1 General

AREMA Manual Chapter 8, Part 23, covers the design, construction, maintenance and inspection of protective systems for railway piers located in and adjacent to channels of navigable waterways.

The purpose of the protective systems is to protect supporting piers of railway structures from damage caused by accidental collision from floating vessels. Such protection should be designed to eliminate or reduce the impact energy transmitted to the pier from the vessel, either by redirection of the force or by absorption, or dissipation of the energy, to non-destructive levels.

The size and type of vessel to be chosen as a basis for design of the pier protection should reflect the maximum vessel tonnage, type of cargo and velocity reasonably to be expected for the specific facility involved.

38.3.2.5.2 Types of Construction

The type of construction to be chosen for the protective system should be based on the physical site conditions and the amount of energy to be absorbed or deflected, as well as the size and ability of the pier itself to absorb or resist the impact.

Some of the more common types of construction are as follows:

- Integral piers - Where the pier is considered to be stable enough to absorb the impact of floating vessels, it may be necessary to attach cushioning devices to the surfaces of the pier in the areas of expected impact to reduce localized damage such as spalling of concrete surfaces and exposure of reinforcing steel, or disintegration of masonry jointing.

- Dolphins - Where depth of water and other conditions are suitable, the driving of pile clusters may be considered. Such clusters have the piles lashed together with cable to promote integral action. The clusters should be flexible to be effective in absorbing impact through deflection.

- Cellular dolphins - May be filled with concrete, loose materials or materials suitable for grouting.

- Floating shear booms - Where the depth of water or other conditions precludes the consideration of dolphins or integral pier protection, floating sheer booms may be used. These are suitably shaped and positioned to protect the pier and are anchored to allow deflection and absorption of energy. Anchorage systems should allow for fluctuations in water level due to stream flow or tidal action.

- Hydraulic devices - Such as suspended cylinders engaging a mass of water to absorb or deflect the impact energy may be used under certain conditions of water depth or intensity of impact. Such cylinders may be suspended from independent caissons, booms projecting from the pier or other supports. Such devices are customarily most effective in locations subject to little fluctuations of water levels.

- Fender systems - Constructed using piling with horizontal wales, is a common means of protection where water depth is not excessive and severe impacts are not anticipated.

- Other types of various protective systems have been successfully used and may be considered by the Engineer. Criteria for the design of protective systems cannot be specified to be applicable to all situations. Investigation of local conditions is required in each case, the results of which may then be used to apply engineering judgment to arrive at a reasonable solution.

38.4 Overpass Structures

Highway overpass structures are placed when the incidences of train and vehicle crossings exceeds certain values specified in the *Facilities Development Manual* (*FDM*). The separation provides a safer environment for both trains and vehicles.

In preparing the preliminary plan which will be sent to the railroad company for review and approval several items of data must be determined.

- Track Profile - In order to maintain clearances under existing structures when the track was upgraded with new ballast, the railroad company did not change the track elevation under the structure causing a sag in the gradeline. The track profile would be raised with a new structure and the vertical clearance for the structure should consider this.

- Drainage - Hydraulic analysis is required if any excess drainage will occur along the rail line or into existing drainage structures. Deck drains shall not discharge onto railroad track beds.

- Horizontal Clearances - The railroad system is expanding just as the highway system. Contact the railroad company for information about adding another track or adding a switching yard under the proposed structure.

- Safety Barrier – The Commissioner of Railroads has determined that the Transportation Agency has authority to determine safety barriers according to their standards. The railroad overpass parapets should be designed the same as highway grade separation structures using solid parapets (Type "SS" or appropriate) and pedestrian fencing where required.

38.4.1 Preliminary Plan Preparation

Standard for Highway over Railroad Design Requirements shows the minimum dimensions for clearances and footing depths. These should be shown on the Preliminary Plan along with the following data.

- Milepost and Direction - Show the railroad milepost and the increasing direction.

- Structure Location - Show location of structure relative to railroad right of way. (Alternative is to submit Roadway Plan).

- Footings - Show all footing depths. Minimum footing depth requirements are shown on the Standard for Highway over Railroad Design Requirements.

- Drainage Ditches - Show ditches and direction of flow.

- Utilities - Show all utilities that are near structure footings and proposed relocation is required.

- Crash Protection – See Standard for Highway over Railroad Design Requirements for crash protection requirements. On a structure widening a crashwall shall be added if the multi-columned pier is equal to or less than 25 feet from centerline of track.

- Shoring – If shoring is required, use a General Note to indicate the location and limit.

- Limits of Railroad Right-of-Way – The locations are for reference only and need not be dimensioned.

38.4.2 Final Plans

The Final Plans must show all the approved Preliminary Plan data and be signed and/or sealed by a Registered Engineer.

38.4.3 Shoring

Railroad companies are particularly concerned about their track elevations. It is therefore very important that shoring is used where required and that it maintains track integrity.

38.4.4 Horizontal and Vertical Clearances

38.4.4.1 Horizontal Clearance

The distance from the centerline of track to the face of back slopes at the top of rail must not be greater than 20’-0” since federal funds are not eligible to participate in costs for providing greater distances unless required by site conditions. Minimum clearances to substructure units are determined based on site conditions and the character of the railroad line. Consideration must be given to the need for future tracks. Site specific track drainage requirements and possible need for an off-track roadway must also be considered.

38.4.4.2 Vertical Clearance

Section 192.31, Wisconsin Statutes requires 23’-0” vertical clearance above top of rail (ATR) for new construction or reconstruction, unless the Office of the Commissioner of Railroads approves less clearance. As a result, early coordination with the Railroads and Harbors Section is required.

Double stack containers at 20’-2” ATR are the highest equipment moving in restricted interchange on rail lines which have granted specific approval for their use. Allowing for tolerance, this equipment would not require more than 21’-0” ATR clearance. Railroad companies desire greater clearance for maintenance purposes and to provide allowance for possible future increases in equipment height.

38.4.4.3 Compensation for Curvature

Where a horizontal clearance obstruction is within 80 feet of curved track AREMA specifications call for lateral clearance increases as stated in *AREMA Manual* Chapter 28, Table 28-1-1.

38.4.4.4 Constructability

The minimum clearances discussed are to finished permanent work. Most railroad companies desire minimum temporary construction clearances to forms, falsework or track protection of 12’- 0” horizontal and 21’-0” vertical. The horizontal clearance provides room for a worker to walk along the side of a train and more than ample room for a train worker who may be required to ride on the side of a 10’-8” wide railroad car. Where piers are to be located close to tracks the type of footing to be used must be given careful consideration for constructability. The depth of falsework and forms for slab decks may also be limited by temporary vertical clearance requirements.

Table of Contents

39.1 General

Signing is an integral part of the highway plan and as such is developed with the roadway and bridge design. Aesthetic as well as functional considerations are essential to sign structure design. Supporting sign structures should exhibit clean, light, simple lines which do not distract the motorist or obstruct view of the highway. In special situations sign panels may be supported on existing or proposed grade separation structures in lieu of an overhead sign structure. Aesthetically this is not objectionable if the sign does not extend below the girders or above the top of the parapet railing. Some of the more common sign support structures are shown in the following figure.

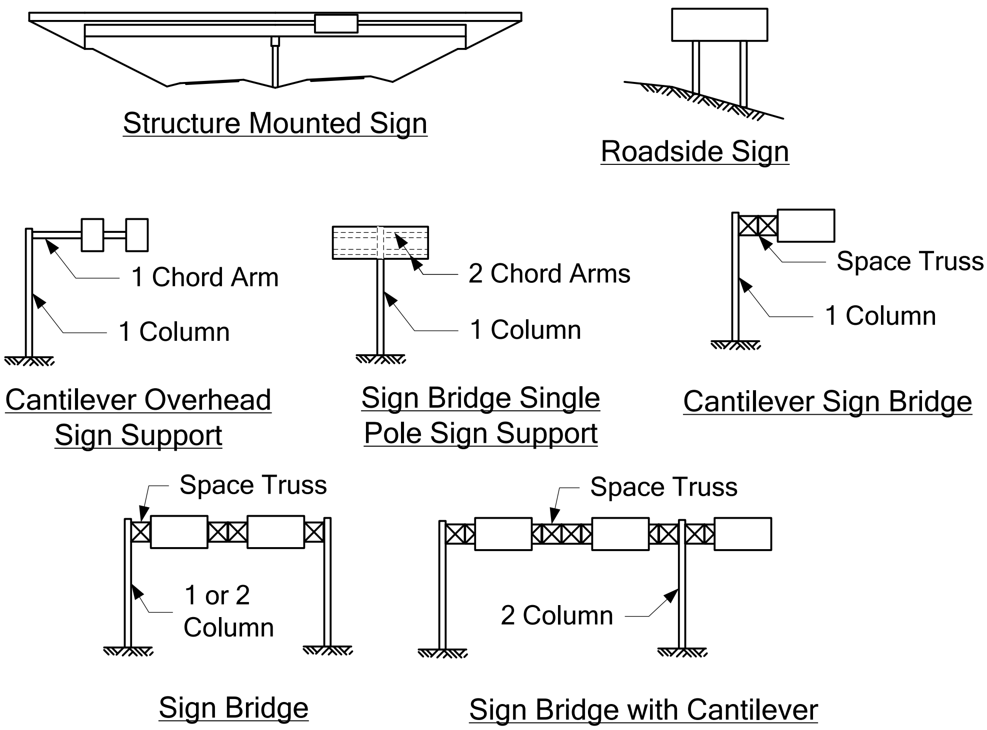

Figure 39.1-1
Sign Support Structures

39.1.1 Signs on Roadway

Roadside sign supports are located behind existing or planned guardrail as far as practical from the roadway out of the likely path of an errant vehicle. If roadside signs are located within the 30 foot corridor and not protected, break-away sign supports are detailed. Wisconsin has experienced that the upper hinge on ground mounted signs with break-away supports does not work and it is not used. Since FHWA has not approved this removal, the hinge is used on all federal projects. DMS, which includes both dynamic message signs and variable message signs, roadside sign type supports are to be protected by concrete barrier or guardrail. All overhead sign-column type supports are located at the edge of shoulder adjacent to the traveled roadway or placed behind barrier type guardrail. See the *Facilities Development Manual* (FDM) 11-55-20.5 for information on shielding requirements.

When protection is impractical or not desirable, the towers shall be designed with applicable extreme event collision loads in accordance to 13.4.10.

Overhead sign structures, for new and replacement structures only, are to have a minimum vertical clearance of 20’-0” above the roadway for the Oversize/Overweight (OSOW) High Clearance Route and 18’-3” for all other routes. Reference 39.4.2 for additional vertical clearance requirements when catwalk or lighting is designed with a sign bridge. See FDM, Procedure 11-35-1, Attachment 1.9, for clearances relating to existing sign structures. The minimum vertical clearance is set slightly above the clearance line of the overpass structure for signs attached directly to the side of a structure.

39.1.2 Signs Mounted on Structures

Signs are typically installed along the major axes of a structure. Wisconsin has allowed sign attachment up to a maximum of a 20 degree skew. Any structure with greater skew requires mounting brackets to attach signs perpendicular to the roadway.

39.1.2.1 Signs Mounted on the Side of Structures

In addition to aesthetic reasons, signs attached to the side of bridge superstructures and retaining walls are difficult to inspect and maintain due to lack of access. Attachment accessories are susceptible to deterioration from de-icing chemicals, debris collection and moisture; therefore, the following guidance should be considered when detailing structure side mounted signs and related connections:

1. Limit the sign depth to a dimension equal to the bridge superstructure depth (including parapet) minus 3 inches.

2. Provide at least two point connections per supporting bracket.

3. Utilize cast-in-place anchor assemblies to attach sign supports onto new bridges and retaining walls.

4. Galvanized or stainless steel adhesive concrete masonry anchor may be used to attach new signs to the vertical face of an existing bridge or retaining wall for shear load application only. Overhead installation is not allowed. Reference 40.16 for applicable concrete masonry anchor requirements.

39.1.2.2 Overhead Structure Mounted Signs

Span deflections of the superstructure due to vehicle traffic are felt in overhead sign structures mounted on those bridges. The amount and duration of sign structure deflections is dependent on the stiffness of the girder and deck superstructure, the location of the sign on the bridge, and the ability of the sign structure to dampen those vibrations out; among others. These vibrations are not easily accounted for in design and are quite variable in nature. For these reasons, the practice of locating overhead sign structures onto bridges should be avoided whenever possible.

The following general guidance is given for those instances where locating a sign structure onto a bridge structure is unavoidable, which may be due to the length of the bridge, or a safety need to guide the traveling public to upcoming ramp exits or into specific lanes on the bridge.

1. Locate the sign structure support bases at pier locations.

2. Build the sign structure base off the top of the pier cap.

3. Provide set back of the tower support of the sign structure behind the back face of the parapet to preclude snagging of any vehicle making contact with the parapet.

4. Use single pole sign supports (equal balanced butterfly's) in lieu of cantilevered (with an arm on only one side of the vertical support) sign supports.

5. Consider the use of a Stockbridge type damper in the horizontal truss of these structures.

6. Do not straddle the pier leaving one support on the pier and one support off the pier in the case of skewed substructure units for full span sign bridges.

39.2 Specifications and Standards

Reference specifications for sign structures are as follows:

- AASHTO "*Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, 6th Edition*", and Interim Revisions
- AASHTO "*Standard Specifications for Highway Bridges, 17th Edition*"
- State of Wisconsin "*Standard Specifications for Highway and Structure Construction*"
- ASTM "*Standards of the American Society for Testing and Materials*"
- AWS D1.1 Structural Welding Code (Steel)
- AWS D1.2 Structural Welding Code (Aluminum)

Standard details for full span 4-chord galvanized steel sign bridge, design data and details for galvanized cantilever steel sign truss and footing are given on the Chapter 39 Standard Details.

Standard details for overhead sign support bases are provided in the Standard Detail Drawing (SDD) sheets of the FDM.

Standard design data and details for break-away sign supports and sign attachment are given on the A Series of the Sign Plate Manual.

39.3 Materials

Wisconsin has historically specified API Spec. 5L, grade 42 pipe as the primary material for the design of sign bridge chords and columns. However, due to supply shortage, API Spec. 5L, grade 46 and 52, ASTM A500 grades B and C, and ASTM A53 grade B types E and S round HSS or pipe (tubular shapes) are allowed as alternate materials for sign bridge truss main members (chords and columns) less than 10 inches diameter. API Spec. 5L, grade 42 remains the preferred material for single column on both full span and cantilever sign bridges due to the toughness requirement to address weldability, fatigue concerns and the non-redundant nature of these structures. Thus, a stricter product specification level 2 (PSL-2) is required. Contractor may substitute grades 46 and 52 steel with the same section properties and product specification requirement for grade 42 pipe at no additional cost to the department. All plates, bars and structural angles shall be ASTM A709 grade 36. ASTM A595 grade A, A572, and A1011 have been used by manufacturers to design round, tapered steel members for overhead sign support arms and uprights. When tubular shapes are used for overhead sign supports, they shall conform to the sign bridge requirements. Unless noted otherwise in the contract plans, all field bolted connections for sign structures shall be made with direct tension indicating (DTI) washers meeting the applicable requirements of high strength A325 bolts as stated in 24.2. More details can be found in the Standard Drawings and Standard Specifications Section 641.

WisDOT policy item:

Installation of flat washers in between faying surfaces of mast arm connection plates are not allowed.

39.4 Design Considerations

39.4.1 Signs on Roadway

Supports for roadside signs are of three types, depending upon the size and type of the sign to be supported. For small signs, the column supports are treated timber embedded in the ground. For larger type I signs and DMS, the columns shall be galvanized steel supported on cylindrical concrete footings. Currently, all steel column supports for roadside type I signs are designed to break-away upon impact, while DMS supports are protected and designed without a break-away system.

WisDOT policy item:

Type I break-away sign supports and foundations are design in accordance to the "AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals 1985". Standard design and support estimates are given in the A3 Series of the "Sign Plate Manual"

Wisconsin does not have standard design or details available for DMS roadside sign supports. Each support structure to be design by structure engineer, and the design must be in compliance with the applicable specifications listed in 39.2. An allowable soil pressure of 3.0 ksf shall be used to design the footings, unless subsurface condition is in question then investigation per 39.6.3 would be implemented to gather necessary design information. DMS sign supports and footings to be detailed with the Structure Plan Section of the contract.

39.4.2 Overhead Sign Structures

Sign structures for support of overhead signs consist of "sign bridges" and "overhead sign supports". Sign bridges are to be either a single column cantilever or butterfly, or a space truss sign bridge supported by one or two columns at each end. For cantilever sign bridge structures, the footing is a single cylindrical shaft with wings to prevent the overturning and twisting of the structure. For space trusses having one or two steel columns on an end, the footing is composed of two cylindrical caissons connected by a concrete cross-girder. The top surface of concrete foundations for all sign bridges is to be located 3' above the highest ground line at the foundation. Occasionally, some sign bridge columns are mounted directly on top of modified bridge parapets, pier caps and concrete towers instead of footings.

Sign bridges also include sign support members mounted directly onto structures. Sign attachments, such as galvanized steel I-beams and/or brackets, typically are anchored to the side of the bridge superstructure. A cantilever truss attached to the side of retaining walls (without a vertical column) is also common.

Similar to sign bridges, all overhead sign supports have single galvanized steel column supported on a cylindrical caisson footing or on top of bridge elements. Cross members can be one chord (monotube), two chord without web elements, or planar truss in either cantilever or full span structure.

The following design data is employed for designing steel sign bridges and overhead sign supports.

Wind Velocity = 90 mph based on the 3-second gust wind speed map and its corresponding methods to find wind pressure.

Dead Load = Wt. of Sign, supporting structure, catwalk, railings and lights.

Ice Load = 3 psf to one face of sign and around surface of members.

Group Load	Load Combination	% of Allowable Stress
I	DL	100
II	DL + W	133
III	DL + Ice + (1/2)W[a]	133
IV	Fatigue	[c]

Table 39.4-1
Group Load Combinations

[a] Minimum Wind Load = 25 psf

[c] See Fatigue section of AASHTO for fatigue loads and stress range limits.

Wind Components	Normal	Transverse
Combination 1	1.0	0.2
Combination 2	0.6	0.3

Table 39.4-2
Wind Components

WisDOT policy item:

Fatigue group loads application is exempt on four chord full span sign bridges supporting type I and II signs mounted on concrete footings as detailed on Standards 39.02 and 39.03. The exemption is also applied to full span overhead sign supports mounted on top of standard concrete bases.

Steel cantilevered sign bridge structures (four chord structures carrying type I and II signage) detailed on Standards 39.10 thru 39.12 are classified, for purposes of fatigue design, as Category 1 structures. These cantilevered support structures are designed to resist Natural Wind Gust and Truck-Induced Gust wind effects, but not designed for Galloping wind effects due to the substantial stiffness and satisfactory performance history in this state. The design of these structures are in accordance to the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 4th Edition" and interim Revisions.

All other sign structures shall be designed with applicable design specifications as stated in 39.2.

Steel cantilever sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. Columns are made from pipe sections. The minimum thickness for the members is indicated on the steel cantilever Standard detail.

Steel full span sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. The minimum thickness of steel web members is 3/16 inch and 0.216 inches for chord members. The connections of web members to chords are designed for bolting or shop welding to allow the contractor the option to either galvanize individual members or complete truss sections after fabrication. The columns are either steel pipe or tubular shape sections with web members (planar truss), see 39.3 for additional details. Steel base plates are used for anchor rod support attachment.

When butt welding round sections, a back-up plate is required since the plates can only be welded from one side. The plate must be of adequate width for film to be used during weld inspection. The exposed weld is ground smooth for appearance as well as fatigue. Shop splices typically done with the use of butt weld, but quality on large weld is difficult to achieve and not economical. Therefore, designers are advised to limit weld size to 5/8", and avoid shop splice on single column member whenever possible.

Aluminum sign bridges are currently not being designed for new structures. Rehabilitation and repair type work may require use of aluminum members and shall be allowed in these limited instances. The following guidelines apply to aluminum structures in the event of repair and rehabilitation type work.

Aluminum sign bridge trusses are designed and fabricated from tubular shapes shop welded together in sections. The minimum thickness of truss chords is ¼ inch and the minimum outside diameter is 4 inches. The recommended minimum ratios of "d/D" between the outside diameters "d" of the web members and "D" of chord members is 0.4. A cast aluminum base plate is required to connect the aluminum columns to the anchor rods. AASHTO Specifications require damping or energy absorbing devices on aluminum overhead sign support structures to prevent vibrations from causing fatigue failures. Damping devices are required before and after the sign panels are erected on all aluminum sign bridges. Stock-bridge type dampers are recommended.

Install permanent signs to sign structures at the time of erection. If the signs are not available, install sign blanks to control vibration. For sign bridges, blanks are attached to a minimum of one-fourth the truss length near its center. The minimum depth of the blanks is equal to the truss depth plus 24 inches. The blanks are to be installed to project an equal dimension beyond the top and bottom chord members. Overhead sign support blanks are equal to the same sizes and at the same locations as the permanent signs. Contact BOS Structures Design Section at 608-267-2869 for further guidance on other vibration controlling methods.

Do not add catwalks to new sign bridges unless they contain DMS over traffic. Catwalks add additional cost to a structure and present a maintenance issue. They can be added if a decision is made to light the signs in the future. Design structures with type I and II signs for a 2'-0" additional (total of 22'-0" for the OSOW High Clearance Route and 20'-3" for all others) vertical clearance when they are located in a continuous median freeway lighting area, for new and replacement sign bridges only. Structures with DMS may require larger vertical clearance to the bottom of the sign depending on the type of catwalk being designed for future installation. The sign bridge should be structurally designed to support a catwalk for those cases when the additional clearance is provided for possible future attachment. Additional accommodations for potential future lighting include providing hand holes in the columns, rodent screens and conduits in the concrete bases.

For structures that are not located in continuous median freeway lighting areas or do not contain DMS, the additional structure height should not be utilized. Therefore, new and replacement sign bridge vertical clearance should be 20'-0" for the OSOW High Clearance Route and 18'-3" for all other routes. No hand holes, rodent screens or conduits shall be installed on the structure in this case. However, all DMS sign bridges require hand holes, rodent screens and conduits.

Brackets, if required, for maintenance of light units are required to support a 2'-3" wide catwalk grating and a collapsible aluminum handrail. Brackets and handrailing for type I and II signs are fabricated from aluminum sections, whereas DMS support brackets are made of galvanized steel. Catwalk grating and toe plates are fabricated from steel and shall be galvanized.

Contract plans shall include details and notes indicating if hand holes are required on one or both towers of the sign bridge.

Overhead sign supports are typically not lit, nor do they require sign maintenance. Therefore, do not detail a catwalk on this type of structure. Also do not detail hand holes, rodent screens and conduits unless the structure is designed to carry an LED changeable message sign, traffic signals or luminaires.

Design of all Sign Bridge structures should reflect some provision for the possibility of adding signs in the future (additional sign area). Consideration should include the number of lanes, possible widening of roadway into the median or shoulder areas, and use of diagrammatic signs to name a few. The truss design should reflect sizing the chords for maximum force at the center of the span. The design of the tower columns and truss webs should allow for signs being placed (say sometime in the future) more skewed to one side than the other. Columns should be selected the same size (outside diameter x thickness) for each side and the design shall reflect different lengths on either side as required by site conditions.

The design sign area and maximum sign depth dimensions for type I and II signs shall be explicitly listed with the design data in the contract plans. Use 3 psf dead load for these types of signs. Provide manufacturer overall DMS dimensions in the plans along with the total weight of the signs. Other loads such as Catwalks, lights and associated attachments must also be included in the overall design data in the contract plans.

The following guidance is recommended for estimating design sign areas.

1. Type I and II signs on full span sign bridges, design sign area equals the largest value resulting from the four requirements below:

    a. Total actual sign area.

b. Two (2) times the controlling tower tributary sign area. Tributary area is computed based on the application of the lever rule on a simply supported truss.

c. Twelve (12) times the number of lanes times the maximum sign depth. The number of lanes is defined as the clear roadway width (including median and shoulders) divided by 12 and rounding down to the nearest whole number.

d. Maximum sign depth times 60% of the span length (center to center of tower).

For design purposes, the standard sign depth shall be limited between 12'-0" and 16'-0". Therefore, vertical clearance and column lengths are to be sized with sign depth not less than 12'-0", unless requested otherwise in the structure survey report. Mega projects with series of sign bridges may deviate from the above requirements provided that coordination is made with the BOS Structures Design Section.

2. Type I and II maximum design sign area for galvanized steel cantilever sign truss is detailed on the Standard for Galvanized Steel Cantilever Sign Truss. Sizing the column length and vertical clearance with 12'-0" sign depth is recommended for future accommodation.

3. DMS sign bridges should be designed with the actual sign dimensions in addition to those of type I and II signs and catwalk as applied.

4. Overhead sign supports are generally designed with the actual sign dimensions and locations. Exception to the approach may be granted to structures with anticipated change in signage.

39.5 Structure Selection Guidelines

Sign structures are composed of "sign bridges" and "overhead sign supports". Either type of sign structure can be configured to be a cantilever sign structure (one column to a horizontal truss arm) or a full-span sign structure (two towers, one on each end of the span). Single pole (butterfly) is another type of sign bridge (chords centered on a single column). Roadside sign supports are an exception to the above naming convention.

39.5.1 Sign Bridges

Sign bridges generally carry type I and II signs, and occasionally DMS. These are large sign structures with sign depths ranging from 5'-0" or less to 18'-0" in the case of large diagrammatic signs. Butterfly sign bridges are limited to 218 sq. ft. of sign area per side. Total sign areas accommodated are up to 264 sq. ft. on cantilever sign bridges. Total sign areas accommodated on full span sign bridges range from 250 to over 1000 sq. ft. These ranges are for approximate guide only. Butterfly sign bridges consisted of either a single chord, or double chord without web members. Other sign bridges generally have truss members consisting of four round chord and angle web members supporting signs on the span or arm (although some three chord structures have been used for full span sign bridges). Towers are comprised of one column for a butterfly, cantilever and full span three chord sign bridges. Full span four round chord sign bridge towers usually consist of two columns joined by angle web members at each end of the span. All "Sign Bridges" are designed by the Bureau of Structures or a consultant. Structure contract plans provide full details that a fabricator can construct the sign bridge from. Standard details for the full-span four chord sign bridge associated with this Chapter of the WisDOT Bridge Manual require a design for each sign bridge structure including foundations. Standard design and details for steel cantilever sign bridge and footing are available for use without performing individual design if a structure meets the limitations required by the standards. These details are used for type I and II sign applications only.

Sign bridges carrying DMS require special consideration. Special concerns include:

1. Size and weight of the sign panel, and attachment location with respect to the axis of the truss.
2. Size and weight of catwalk, and attachment location with respect to the axis of the truss.
3. Consideration of wind effects unique to these signs.
4. Modification to support brackets. All catwalk and sign bracket connections shall be made with friction type connections and high strength A325 bolts with DTI washers.

Wisconsin recommends the use of the Minnesota four chord steel angle truss configuration for sign bridges carrying DMS, providing that the designer checks the design of each member and connection details and make necessary modification to conform to the latest AASHTO Standard specification requirements as stated in 39.2. Each foundation shall be designed and included in the contract plans with the sign bridge structures.

39.5.2 Overhead Sign Supports

Overhead sign supports are smaller sign structures carrying type II (smaller) directional signs, limited amounts of type I signs and small LED or changeable message signs. Type II sign depths have ranged from 3’-0” to 4’-0” deep for traffic directional signs, and up to 10’-0” for small information type I signs. When a sign is larger than 10’-0” deep, the structure is to be designed as a sign bridge. Cantilever overhead sign supports accommodated up to 45 sq. ft. of sign area. Total sign areas accommodated on full span overhead sign supports range up to 300 sq. ft. These ranges are again an approximate guide and can be more or less depending on variables such as span length, location of the sign with respect to the tower(s) the height of the tower(s), etc. Towers are comprised of single column (uniform or tapered pipe) for either the cantilever or full span overhead sign support. Arms on cantilever or the span on a full span overhead sign support are either one chord (uniform or tapered pipe), or two chords with or without angle web members depending on the span length and sign depth. Due to the variability of factors that can influence the selection of structure type, designers are encouraged to contact BOS Structures Design Section for further assistance when sign areas fall outside of the above limits, or when structural geometry is in question. “Overhead Sign Supports” are normally bid by contractor and designed by a fabricator or by another party for a fabricator to construct. Typical structures with steel poles on standard concrete bases usually have the least plan detail associated with them and are normally depicted in the Construction Detail portion of the state contract plans. However, it is recommended that plan development for projects with multiple structures, such as major or mega projects, and structures mounted on non-standard supports to be prepared by structural engineers and placed in section 8 of the contract plans along with the sign bridge plans. When a standard concrete base design is required the corresponding SDD sheets shall be used as drawings, and they must be inserted into the contract plans for overhead sign supports. See the WisDOT FDM Procedures 11-55-20 and 15-1-20 for more information on “Overhead Sign Supports”.

39.6 Geotechnical Guidelines

Several potential problems concerning the required subsurface exploration for foundations of sign structures exist. These include:

- The development and location of these structures are not typically known during the preliminary design stage, when the majority of subsurface exploration occurs. This creates the potential for multiple drilling mobilizations to the project.
- Sometimes these structures are located in areas of proposed fill soils. The source and characteristics of this fill soil is unknown at the time of design.
- The unknowns associated with these structures in the scoping/early design stages complicate the consultant contracting process. How much investigation should be scoped in the consultant design contract?

Currently, all sign structure foundations are completely designed and detailed in the project plans. Sign-related design information can be found in the *Facilities Development Manual* (FDM) or Bridge Manual as described in the following sections.

WisDOT policy item:

The length of a cast-in-place shaft foundation shall be limited to 20’-0” for both sign bridges and overhead sign supports. Deviation from this policy item may be allowed provided coordination is made with BOS Structures Design Section.

39.6.1 Sign Bridges

WisDOT has created a standard foundation design for cantilever sign bridges carrying Type I and II signs. This standard foundation is presented on the Standard for Cantilever Truss Footing. The wings on this single shaft footing are used to help resist torsion. If a cantilever sign bridge exceeds the criteria/limitations (shown on the Standard for Galvanized Steel Cantilever Sign Truss), the standard foundation shall not be utilized, and an individual foundation must be fully designed. This customized design will involve determining the subsurface conditions as described in section 39.6.3.

Foundations supporting all butterfly and full span sign bridges are custom designed. They generally have two cylindrical shafts connected by a concrete cross-girder below the columns. Other foundations such as single shaft, pile foundation and spread footing may be detailed when subsurface condition, constructability issue or economic present a more desirable design. WisDOT has no standard details for the foundations of these structures.

39.6.2 Overhead Sign Supports

Overhead sign supports are described in Sections 11-55-20 and 15-1-20 of the FDM. In addition, Section 641 of the Standard Specifications outlines the design/construction aspects of these structures.

If these structures are carrying type I and II signs, and meeting several criteria/limitations that are listed on the SDD's, the designer can use WisDOT-developed standard foundations for them. The designer can then insert the proper SDD sheet into the plans. SDD sheets exist for cantilever overhead sign supports. These single shaft bases for cantilever overhead sign supports vary in depth and range from 24" to 42" in diameter (SDD 15c22-2 thru 15c25-2). Another SDD sheet applies to full-span overhead sign supports and is 36" in diameter (SDD 15c15-3). The standard foundations in these SDD sheets were designed using slightly conservative soil design parameters. If the design criteria for these standard designs are not met, the SDD sheets cannot be used, the structure foundation must be fully designed and the unique details shall be done in accordance to the overhead sign support mounted on non-standard supports procedure described in 39.5.2. This involves determining the subsurface conditions as described in the following section.

39.6.3 Subsurface Investigation and Information

No subsurface investigation/information is necessary for any of the sign structures that meet the limitations for allowing the use of WisDOT standard foundations. Appropriate subsurface information is necessary for any of these structures that require custom designs.

There may be several methods to obtain the necessary subsurface soil properties to allow for a custom design of foundations, as described below:

- In areas of fill soils, the borrow material may be unknown. The designer should use their best judgment as to what the imported soils will be. Standard compaction of this material can be assumed. Conservative soil design parameters are encouraged.

- The designer may have a thorough knowledge of the general soil conditions and properties at the site and can reasonably estimate soil design parameters.

- The designer may be able to use information from nearby borings. Judgment is needed to determine if the conditions present in an adjacent boring(s) are representative of those of the site in question.

- If the designer cannot reasonably characterize the subsurface conditions by the above methods, a soil boring and Geotechnical report (Site Investigation Report) should be completed. Necessary soil design information includes soil unit weights, cohesions, phi-angles and location of water table.

Designers, both internal and consultant, should also be aware of the potential of high bedrock, rock fills and the possible conflict with utilities and utility trenches.

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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

40.2 History

40.2.1 Concrete

Prior to 1975, all concrete structures were designed by the Service Load Method. The allowable design stress was 1400 psi based on an ultimate strength of 3500 psi except for deck slabs. In 1965, the allowable stress for deck slabs on stringers was reduced from 1400 to 1200 psi. The reason for the change was to obtain thicker concrete decks by using lower strength. The thicker deck was assumed to have more resistance to cracking and greater durability. During this time no changes were made in the physical properties of the concrete or the mixing quantities until 1974 when more cement was added to the mix for concrete used in bridge decks.

In 1980, the use of set retarding admixtures was required when the atmospheric temperature at the time of placing the concrete is 70°F or above; or when the atmospheric temperature is 50°F or above and it is expected that the time to place the concrete of any span or pour will require four or more hours. Retarding admixtures reduce concrete shrinkage cracking and surface spalling. Also, during the early 1980's a water reducing admixture was provided for Grades A and E concrete mixes to facilitate workability and placement.

40.2.2 Steel

Prior to 1975, Grade 40 bar steel was used in the design of reinforced concrete structures. This was used even though Grade 60 bars may have been furnished on the job. Allowable design stress was 20 ksi using the Service Load Method and 40 ksi using the Load Factor Method.

40.2.3 General

In 1978, Wisconsin Department of Transportation discovered major cracking on a two-girder, fracture critical structure, just four years after it was constructed. In 1980, on the same structure, major cracking was discovered in the tie girder flange of the tied arch span.

This is one example of the type of failures that transportation departments discovered on welded structures in the 1970's and '80's. The failures from welded details and pinned connections led to much stricter standards for present day designs.

All areas were affected: Design with identification of fatigue prone details and classifications of fatigue categories (AASHTO); Material requirements with emphasis on toughness and weldability, increased welding and fabrication standards with licensure of fabrication shops to minimum quality standards including personnel; and an increased effort on inspection of existing bridges, where critical details were overlooked or missed in the past.

Wisconsin Department of Transportation started an in-depth inspection program in 1985 and made it a full time program in 1987. This program included extensive non-destructive testing. Ultrasonic inspection has played a major role in this type of inspection. All fracture critical structures, pin and hanger systems, and pinned connections are inspected on a five-year cycle now.

40.3 Bridge Replacements

Bridge rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. In order to obtain federal funding eligibility for rehabilitation or replacement; the bridge must be Structurally Deficient or Functionally Obsolete. The Federal Sufficiency Number is a guide for federal participation which is required to be less than 50 for replacement. Also, Wisconsin DOT requires the Rate Score to be less than 75. Bridges are not eligible for replacement unless the Substructure or Superstructure Condition is 4 or less or the Inventory Rating is less than HS10 or the Alignment Appraisal is 4 or less.

A bridge becomes Structurally Deficient when the condition of the deck, superstructure or substructure is rated 4 or less; or when the inventory load capacity is less than 10 tons (89.0 kN); or when the waterway adequacy is rated a 2.

A bridge becomes Functionally Obsolete when the bridge roadway width, vertical clearance, or approach alignment is substandard (appraisal rating of 3 or less), or when the inventory load capacity is less than 15 tons; or when the waterway adequacy is rated a 3 or less.

See FDM 11-40-1, 1.5 for policies regarding necessary bridge width and structural capacity to determine eligibility for bridge rehabilitation* versus bridge replacement.

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of insuring some level of acceptable serviceability. The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are sufficient to safely carry present and projected traffic. Information necessary to determine structure sufficiency includes structure inspection, inventory, traffic, maintenance, capacity and functional adequacy. The methods of reconstruction are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to insure that rehabilitation will remove all structural deficiencies. FHWA requires this review and Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation. On high cost bridges, a closer check of the Functionally Obsolete Criteria may be required. On high cost bridges a 2' shoulder is acceptable on a low speed, low volume roadway having a good accident record. After rehabilitation work is completed, the bridge should not be Structurally Deficient or Functionally Obsolete. A sufficiency number greater than 80 is also required after completion of the rehabilitation work. However, if conditions exist that would prevent the completed improvement from correcting all deficiencies, WisDOT shall determine if the proposed project is eligible based on safety and the public interest. Contact the Bureau of Structures Development Section for a waiver of the sufficiency number requirement.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic.

Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and riding surface. Wisconsin has experienced some cracking in the concrete overlays and parapets during rehabilitation construction. The apparent cause of cracking is related to traffic impact due to rough structure approaches. Particular attention should be given to providing a smooth transition on temporary approaches and over expansion joints on all bridge decks during rehabilitation. Other causes of cracking are related to shrinkage, temperature, wind induced vibrations, concrete quality and improper curing.

Options for deck rehabilitation are as follows:

1. Asphalt Patch
2. Asphalt or Polymer Modified Asphaltic Overlay
3. Concrete or Modified Concrete Patch
4. Waterproof Membrane with an Asphalt Overlay (currently not used)
5. Concrete Overlay - Grade E Low Slump Concrete, or Micro-Silica Modified Concrete

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project
    a. Deck condition equal 4 or 5 and;
    b. Wear course or wear surface less than or equal to 3.
    c. No roadway work scheduled for at least 3 years.
2. Interstate Bridge with Roadway Work
    a. No previous work in last 10 years or;
    b. Deck Condition less than or equal 4.
    c. Wear course or wear surface less than or equal to 4.
3. Rehab not needed on Interstate Bridges if:
    a. Deck rehab work less than 10 years old.
    b. Deck condition greater than 4.
    c. Wear surface or wear course greater than or equal 4.

4. All Bridges

<u>WisDOT policy item:</u>

On major rehab work, build to current standards such as safety parapets, full shoulder widths, etc. Use the current Bridge Manual standards and tables. Exceptions to this policy require approval from the Bureau of Structures Development Section.

   a. Evaluate cost of repeated maintenance, traffic control as well as bridge work when determining life-cycle costs.

   b. Place overlays on all concrete superstructure bridges if eligible.

   c. For all deck replacement work the railing shall be built to current standards.

5. All Bridges with Roadway Work

   Coordinate with the Region the required staging of bridge related work.

A number of specific guidelines are defined in subsequent sections. As with any engineering project, the engineer is allowed to use discretion in determining the applicability of these guidelines.

40.5 Deck Overlays

If the bridge is a candidate for replacement or a new deck, serviceability may be extended 3 to 7 years by patching and/or overlaying the deck with only a 1-1/2" minimum thickness asphaltic mat on lightly traveled roadways. Experience indicates the asphalt tends to slow down the rate of deterioration while providing a smooth riding surface. However, these decks must be watched closely for shear or punching shear failures as the deck surface problems are concealed.

For applications where the deck is structurally sound and service life is to be extended there are other methods to use. A polymer modified asphaltic overlay may be used to increase deck service life by approximately 15 years. If the concrete deck remains structurally sound, it may be practical to remove the existing overlay and place a new overlay before replacing the deck.

A 1-1/2" concrete overlay is expected to extend the service life of a bridge deck for 15 to 20 years. On delaminated but structurally sound decks a concrete overlay is often the only alternative to deck replacement. Prior to placing the concrete overlay, a minimum of 1" of existing deck surface should be removed. On all bridges low slump Grade E concrete is the specified standard with close inspection of concrete consolidation and curing. If the concrete deck remains structurally sound; it may be practical to remove the existing overlay and place a second deck overlay before replacing the entire deck. After the concrete overlay is placed, it is very important to seal all the deck cracks. Experience shows that salt water passes thru these cracks and causes deterioration of the underlying deck.

On deck overlays preparation of the deck is an important issue after removal of the top surface. Check the latest Special Provisions and/or specifications for the method of payment for Deck Preparation where there are asphalt patches or unsound concrete.

Micro-silica concretes have been effectively used as an alternate type of concrete overlay. It provides excellent resistance to chloride penetration due to its low permeability. Micro-silica modified concrete overlays appear very promising; however, they are still under experimental evaluation. Latex overlays when used in Wisconsin have higher costs without noticeable improved performance.

Ready mixed Grade E concrete with superplasticizer and fiber mesh have been tried and do not perform any better than site mixed concrete produced in a truck mounted mobile mixer.

Bridges with Inventory Ratings less than HS10 with an overlay shall not be considered for concrete overlays, unless approved by the Bureau of Structures Design Section. Bridges reconstructed with overlays shall have their new Inventory and Operating Ratings shown on the bridge rehabilitation plans. Verify the desired transverse cross slope with the Regions as they may want to use current standards.

40.5.1 Guidelines for Bridge Deck Overlays

As a structure ages, rehabilitation is a necessary part of insuring a level of acceptable serviceability. Overlays can be used to extend the service lives of bridge decks that have surface deficiencies. Guidelines for determining if an overlay should be used are:

- The structure is capable of carrying the overlay deadload;
- The deck and superstructure are structurally sound;
- The desired service life can be achieved with the considered overlay and existing structure;
- The selected option is cost effective based on the structure life.

40.5.2 Deck Overlay Methods

An AC Overlay or Polymer Modified Asphaltic Overlay should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic. All full depth repairs shall be made with PC concrete.

Guidelines for determining the type of deck overlay method to achieve the desired extended service life are:

AC Overlay (ACO): 5 years average life expectancy

- The minimum asphaltic overlay thickness is 1-1/2”.
- The grade change due to overlay thickness can be accommodated at minimal cost.
- Deck or bridge replacement is programmed within 7 years.
- Raising of floor drains or joints is not required.
- Spalls can be patched with AC or PC concrete with minimal surface preparation.

Polymer Modified Asphaltic Overlay: 15 to 20 years life expectancy

- This product may be used as an experimental alternate to LSCO given below. CAUTION – Core tests have shown the permeability of this product is dependent on the aggregate. Limestone should not be used.

Polymer Overlay: 10 to 15 years life expectancy

- A 1/4-inch thick, two layer system comprised of a two-component polymer in conjunction with natural or synthetic aggregates. Use 5 psf for dead load, DW.
- Works well to seal decks and/or provide traction.

The minimum required concrete age is 28 days prior to application, although a longer period of time would allow more initial concrete cracking to occur which the resin would then be able to seal.

AC Overlay with a Waterproofing Membrane (ACOWM): (Currently not used)

Low Slump Concrete Overlay *(LSCO): 15 to 20 years life expectancy

- Minimum thickness is 1-1/2" PC concrete overlay.
- Joints and floor drains will be modified to accommodate the overlay.
- Deck deficiencies will be corrected with PC concrete.
- The prepared deck surfaces will be scarified or shot blasted.
- There is no structural concern for excessive leaching at working cracks.
- Combined distress area is less than 25%.
- May require crack sealing the following year and periodically thereafter.

* Note: Or another PC concrete product as approved by the Bureau of Structures Development Section and coordinated with the Region.

40.5.3 Maintenance Notes

- All concrete overlays crack immediately. If the cracks in the deck are not sealed periodically, the rate of deterioration can increase rapidly.

40.5.4 Special Considerations

- On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans.
- If more than 1/3 of the steel is exposed, either the centers of adjacent spans must be shored or only longitudinally overlay 1/3 of the bridge at a time.

40.5.5 Railings and Parapets

Overlays increase vehicle lean over sloped face parapets resulting in vehicles on bridges with higher ADT and/or speed having an increased likelihood of impact with lights/obstructions on top of, or behind, the parapet. See Chapter 30-Railings for guidance pertaining to railings and parapets associated with rehabilitation structures projects.

Sub-standard railings and parapets should be improved. An example of such a sub-standard barrier would be a curb with a railing or parapet on top. Contact the Bureau of Structures Development Section to discuss solutions.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. Refer to the criteria in 40.3 of this chapter for additional 3R project considerations. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:

Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating greater than HS18, but less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the *Facilities Development Manual* and *FDM SDD 14b7* for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, replace existing intermediate concrete diaphragms with new steel diaphragms at existing diaphragm locations (i.e. don't add intermediate lines of diaphragms). See Chapter 19 Standard Details and Steel Diaphragm Insert Sheets for additional information.

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 45", 54" and 70" girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections' draped and undraped strand patterns.

The 45", 54", and 70" girders in Chapter 40-Bridge Rehabilitation standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at $0.75f_{pu}$,
- A concrete haunch of 2-1/2",
- Slab thicknesses from Chapter 17-Superstructure - General,
- A future wearing surface of 20 psf,
- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads,
- 0.5" or 0.6" dia. strands (in accordance with the Standard Details),
- f'_c girder = 8,000 psi,
- f'_c slab = 4,000 psi, and
- Required f'_c girder at initial prestress < 6,800 psi

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	102	112
6’-6”	100	110
7’-0”	98	108
7’-6”	96	102
8’-0”	94	100
8’-6”	88	98
9’-0”	88	96
9’-6”	84	90
10’-0”	84	88
10’-6”	82	86
11’-0”	78	85
11’-6”	76	84
12’-0”	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	130	138
6’-6”	128	134
7’-0”	124	132
7’-6”	122	130
8’-0”	120	128
8’-6”	116	124
9’-0”	112	122
9’-6”	110	118
10’-0”	108	116
10’-6”	106	112
11’-0”	102	110
11’-6”	100	108
12’-0”	98	104

70” Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	150*	160*
6’-6”	146*	156*
7’-0”	144*	152*
7’-6”	140*	150*
8’-0”	138*	146*
8’-6”	134*	142*
9’-0”	132*	140*
9’-6”	128*	136
10’-0”	126*	134
10’-6”	122	132
11’-0”	118	128
11’-6”	116	126
12’-0”	114	122

Table 40.7-1
Maximum Span Length vs. Girder Spacing

*For lateral stability during lifting these girder lengths will require pick up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.

40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. Also, reference is made to the criteria in 40.3 of this Chapter for additional 3R project considerations. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, consideration shall be given to replacing the entire deck in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the life-cycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W" rather than 54"). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. *The girders used for widenings may be the latest Chapter 19-Prestressed Concrete sections designed to LRFD or the sections from Chapter 40-Bridge Rehabilitation designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.*

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet **LRFD [3.6.5]** (600 kip loading) as a widening is considered rehabilitation. Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development Section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don't add intermediate lines of diaphragms).

40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3' or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading). Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

Approval is required from BOS for all superstructure replacement projects. In order for a superstructure replacement to be allowed, the substructure must meet the criteria outlined below. This justifies the cost of a new superstructure by ensuring a uniform level of reliability for the entire structure.

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 600 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 600 kip vehicular impact loading need not be considered if a minimum of "vehicle protection" is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

For abutments, evaluate the piles, or bearing capacity of the ground if on spread footings, utilizing Service I loading. The abutment body should be evaluated using Strength I loading.

The superstructure shall be designed to current LRFD criteria.

40.10 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.

40.11 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

Existing steel expansion devices shall be modified or replaced with watertight expansion devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6' or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide downhill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

40.14 Superstructure Inspection

40.14.1 Prestressed Girders

On occasion prestressed concrete girders are damaged during transport, placement, or as a result of vehicle impact. Damage inspection and assessment procedures are necessary to determine the need for traffic restrictions, temporary falsework for safety and/or strength. Three predominant assessment areas are damage to prestressing strands, damage to concrete, and remaining structural integrity.

Where damage to a girder results in any significant loss of concrete section, an engineering analysis should be made. This analysis should include stress calculations for the damaged girder with comparison to the original design stresses. These calculations will show the loss of strength from the damage.

Assessment of damage based on the loss of one or more prestressing strands or loss of prestress force is given as reason for restriction or replacement of girders. Some of the more common damages are as follows with the recommended maintenance action. However, an engineering assessment should be made on all cases.

1. If cracking and spalling are limited to the lower flange, patching is usually performed. This assessment should be based on calculations that may allow repair.

2. If cracking continues from flange into web, the girder is normally replaced. Findings indicate that sometimes repair-in-place may be the preferred decision.

3. Termini of cracks are marked, and if the cracks continue to grow the girder may be replaced. This is a good method to determine the effect of loads actually being carried.

4. When large areas of concrete are affected, or when concrete within stirrups is fractured, replace. At times it may be more appropriate to repair-in-place than replace.

Repair and/or replacement decisions are based on structural integrity. Load capacity is by far the most important rationale for selection of repair methods. Service load capacity needs to be calculated if repair-in-place is contemplated.

Evaluations of criteria for assessment of damage related to the two predominant areas associated with structural integrity are the following:

1. A structural analysis is made to determine the stresses in a damaged girder on the basis of the actual loads the girder will carry. This technique may result in a girder that has less capacity than the original, but can still safely carry the actual loads. This assessment results in consideration of possible repair-in-place methods which normally are cost effective.

   or

2. A structural analysis is made to determine the load capacity and rating of the girder. If the capacity and rating of the girder is less than provided by the original design, the girder shall be replaced. This assessment will provide a girder equal to the original design, but precludes possible repair-in-place methods that are normally less costly.

Location and size of all spalled and unsound concrete areas shall be recorded. Location, length, and width of all visible cracks shall be documented. All damage to prestress strands and reinforcing steel shall be reported. Location of hold-down devices in the girder shall be shown in relation to the damage. Horizontal and vertical misalignment along the length of the girder, and at points of localized damage, shall be reported. (These measurements might best be made by string-lining). Growth of cracks shall be monitored to determine that the cracked section has closed before extending to the web.

Critical damage is damage to concrete and/or the reinforcing elements of prestressed concrete girders such as:

1. Cracks extend across the bottom flange and/or in the web directly above the bottom flange. (This indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset is measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.

4. Vertical misalignment in excess of the normal allowable.

5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface. (This indicates permanent deformation of stirrups).

40.14.2 Steel Beams

These are three alternate methods of repairing damaged steel beams. They are:

1. Replace the total beam,

2. Replace a section of the beam, or

3. Straighten the beam in-place by heating and jacking.

The first alternate would involve removing the concrete deck over the damaged beam, remove the damaged section and weld in a new piece; then reconstruct the deck slab and railing over the new girder. Falsework support is required at the locations where the beams are cut and probably in the adjacent span due to an unbalanced condition.

The second alternate involves cutting out a section of the beam after placing the necessary supporting members. The support is placed using calibrated jacks. The section is cut out as determined by the damage. A new section plus any vertical stiffeners and section of cover plates would be welded in. This involves butt welds on both the flange and web. The welding of the web is difficult due to minor misalignments to start with plus the tendency of thin plates to move from the heat of welding.

The third alternate of heating and jacking the in-place beam to straighten it is a difficult procedure but can be done by personnel familiar and knowledgeable of the process. It is important to maintain heat control under 1300°F maximum. Use an optical pyrometer to determine heat temperature. There is no specified tolerance for the straightened member. The process is deemed satisfactory when a reasonable alignment is obtained.

Based on the three alternates available, the estimated costs involved and the resultant restoration of the beam to perform its load carrying capacity, heat straightening is a viable option in many cases.

The structural engineer who will be responsible for plan preparation should field review the site with the Regional Bridge Maintenance Engineer.

40.15 Substructure Inspection

The inspection of substructure components may reveal deteriorating concrete in areas exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows. Footings and pilings exposed due to erosion and undermining could result in loss of bearing capacity and/or section.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Reuse of steel pile sections will require checking the remaining allowable load carrying capacity. Steel piling should be checked immediately below the splash zone or water line for deterioration and possible loss of section. High section loss has occurred in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line. Bearing capacities of existing footings and pilings may have to be recomputed in order to determine if superstructure loading can be safely carried.

Timber substructure components may exhibit deterioration due to fungus decay, abrasion wear and weathering. Also, physical damage may be caused by vermin attacks, chemicals, fires, and collisions. Prior to reuse, timber backed abutments and pier bents shall be checked, by boring, for material and mechanical condition, section loss and structural adequacy.

40.15.1 Hammerhead Pier Rehabilitation

Pier caps and sometimes shafts of these piers have become spalled due to leaky joints in the deck. The spalling may be completely around some of the longitudinal bar steel destroying the bond. However, experience shows that the concrete usually remains sound under the bearing plates. There is no known reason for this except that maybe the compressive forces may prevent salt intrusion or counteract freeze thaw cycles.

If the longitudinal bars are full length, the bond in the ends insures integrity even though spalling may occur over the shaft. Corrective action is required as follows:

1. Place a watertight expansion joint in the deck.
2. Consider whether bearing replacement is required.
3. Analyze the type of cap repair required.
    a. Clean off spalled concrete and place new concrete.
    b. Analyze capacity of bars still bonded to see if unbonded bars are needed. Use ultimate strength analysis.
    c. Consider repair method for serious loss of bar steel capacity.
        i. Add 6” of cover to cap. Add additional bar steel. Grout in U shaped stirrups around bars using standard anchor techniques.

ii. Use steel plates and post-tensioning bars to place compression loads on both ends of cap. Cover exposed bars with concrete.

iii. Pour wing extension under pier caps beginning at base to take all loads in compression. This would alter pier shape.

d. Consider sloping top of pier to get better drainage.

e. Consider placing coating on pier top to resist water intrusion.

If the shaft is severely spalled, it will require a layer of concrete to be placed around it. Otherwise, patching is adequate. To add a layer of concrete:

1. Place anchor bars in existing solid concrete. Use expansion anchors or epoxy anchors as available.

2. Place wire mesh around shaft.

3. Place forms and pour concrete. 6” is minimum thickness.

40.15.2 Bearings

All steel bridge bearings should be replaced as shown in Chapter 27-Bearings. Compressive load and adhesion tests will be waived for steel laminated elastomeric bearings where these bearings are detailed to meet height requirements.

In general replace lubricated bronze bearings with Teflon bearings. If only outside bearings are replaced, the difference in friction factors can be ignored. Where lubricated bronze bearings might be used, following is the design criteria.

For the expansion bearings, two additional plates are employed over fixed bearings, a top plate and a lubricated bronze plate. Current experience indicates that a stainless steel top plate reduces corrosion activity and is the recommended alternate to steel. The top plate is set on top of a lubricated bronze plate allowing expansion and contraction to occur. Laboratory testing of lubricated bronze plates indicated a maximum coefficient of friction varying from 8 to 14 percent for a loading of 200 kips. Current BOS practice for steel girder Type “A” and prestressed girder expansion bearings of employing a 10 percent maximum and a 6 percent minimum friction value for design is in accordance to laboratory test results. For Type “A” bearing details refer to Standard Details.

40.16 Concrete Anchors for Rehabilitation

Concrete anchors are used to connect concrete elements with other structural or non-structural elements and can either be cast into concrete (cast-in-place anchors) or installed after concrete has hardened (post-installed anchors). This section discusses post installed anchors used on bridge rehabilitation projects. Note: this section is also applicable for several cases where post installed anchors may be allowed in new construction.

This section includes guidance based on the ACI 318-14 manual, hereafter referred to as ACI. (AASHTO currently does not have guidance for anchors.)

40.16.1 Concrete Anchor Type and Usage

Concrete anchors installed in hardened concrete, post-installed anchors, typically fall into two main groups – adhesive anchors and mechanical anchors. For mechanical anchors, subgroups include undercut anchors, expansion (torque-controlled or displacement controlled) anchors, and screw anchors.

Mechanical anchors are seldom used for bridge rehabilitations and current usage has been restricted due to the following concerns: anchor installation (hitting rebar, abandoning holes, and testing), the number of different anchor types, design requirements that are more restrictive than adhesive anchors, the ability to remove and reuse railings/fences, and the collection of salt water within the hole. Note: mechanical anchors may be considered when it has been determined cast-in-place anchors or through bolts are cost prohibitive, adhesive anchors are not recommended, and the above concerns for mechanical anchors have been addressed. See post-installed anchor usage restrictions for additional information.

An Approved Products List addresses some of the concerns for creep, shrinkage, and deterioration under load and freeze-thaw cycles for adhesives anchors. Bridge rehabilitations projects typically use adhesive anchors for abutment and pier widenings. Other bridge rehabilitation applications may also warrant the use of adhesive anchors when required to anchor into existing concrete. Refer to the Standards for several examples of anchoring into existing concrete.

In limited cases, post installed concrete anchors may be allowed for new construction. One application is the allowance for the contractor to use adhesive anchors in lieu of cast-in-place concrete anchors for attaching pedestrian railings/fencing. Refer to Chapter 30 Standards for pedestrian railings/fencing connections.

The following is a list of current usage restrictions for post installed anchors:

Usage Restrictions:

- Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers.

- **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column).**

- Adhesive anchors installed in the overhead or upwardly inclined position and/or under sustained tension loads shall not be used.

- The department has placed a moratorium on mechanical anchors. Usage is subject to prior-approval by the Bureau of Structures.

40.16.1.1 Adhesive Anchor Requirements

For adhesive anchors, there are two processes used to install the adhesive. One option uses a two-part adhesive that is mixed and poured into the drilled hole. The second option pumps a two-part adhesive into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. With either process, the hole must be properly cleaned and a sufficient amount of adhesive must be used so that the hole is completely filled with adhesive when the rebar or bolt is inserted. The adhesive bond stresses, as noted in Table 40.16 1, are determined by the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 or ACI 355.4.

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 6 times the anchor diameter. The maximum embedment depth for is 20 times the anchor diameter.

The manufacturer and product name of adhesive anchors used by the contractor must be on the Department's approved product list for "Concrete Adhesive Anchors".

Refer to the *Standard Specifications* for additional requirements.

40.16.1.2 Mechanical Anchor Requirements

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 10 times the anchor diameter. The minimum member is the great of the embedment depth plus 4 inches and 3/2 of the embedment depth. ***Mechanical anchors are currently not allowed.***

40.16.2 Concrete Anchor Reinforcement

Reinforcement used to transfer the full design load from the anchors into the structural member is considered anchor reinforcement. **ACI [17.4.2.9]** and **ACI [17.5.2.9]** provide guidance for designing anchor reinforcement. When anchor reinforcement is used, the design strength of the anchor reinforcement can be used in place of concrete breakout strength per 40.16.3 and 40.16.4. Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load is considered to be supplementary reinforcement.

Per **ACI [2.3],** concrete anchor steel is considered ductile if the tensile test elongation is at least 14 percent and reduction in area is at least 30 percent. Additionally, steel meeting the requirements of ASTM A307 is considered ductile. Steel that does not meet these requirements is considered brittle. Rebar used as anchor steel is considered ductile.

40.16.3 Concrete Anchor Tensile Capacity

Concrete anchors in tension fail in one of four ways: steel tensile rupture, concrete breakout, pullout strength of anchors in tension, or adhesive bond. The pullout strength of anchors in tension only applies to mechanical anchors and the adhesive bond only applies to adhesive anchors. Figure 40.1 shows the concrete breakout failure mechanism for anchors in tension.

The minimum pullout capacity (Nominal Tensile Resistance) of a single concrete anchor is determined according to this section; however, this value is only specified on the plan for mechanical anchors. The minimum pullout capacity is not specified on the plan for adhesive anchors because the anchors must be designed to meet the minimum bond stresses as noted in Table 40.16-1. If additional capacity is required, a more refined analysis (i.e., anchor group analysis) per the current version of ACI 318-14 Chapter 17 is allowable, which may yield higher capacities.

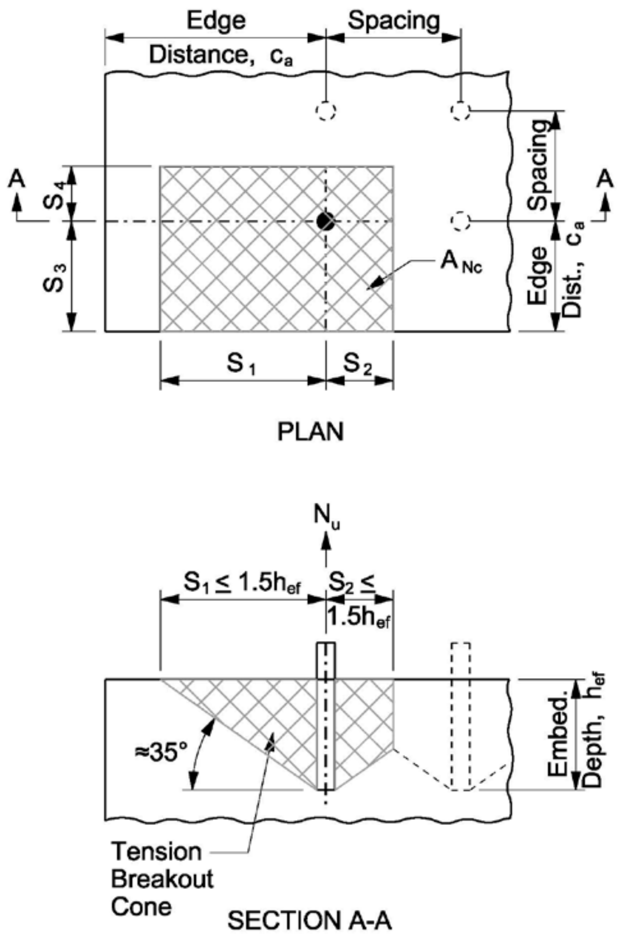

Figure 40.1
Concrete Breakout of Concrete Anchors in Tension

The projected concrete breakout area, A_{Nc}, shown in Figure 40.1 is limited in each direction by S_i:

S_i = Minimum of:

1. 1.5 times the embedment depth (h_{ef}),
2. Half of the spacing to the next anchor in tension, or
3. The edge distance (c_a) (in).

Figure 40.2 shows the bond failure mechanism for concrete adhesive anchors in tension.

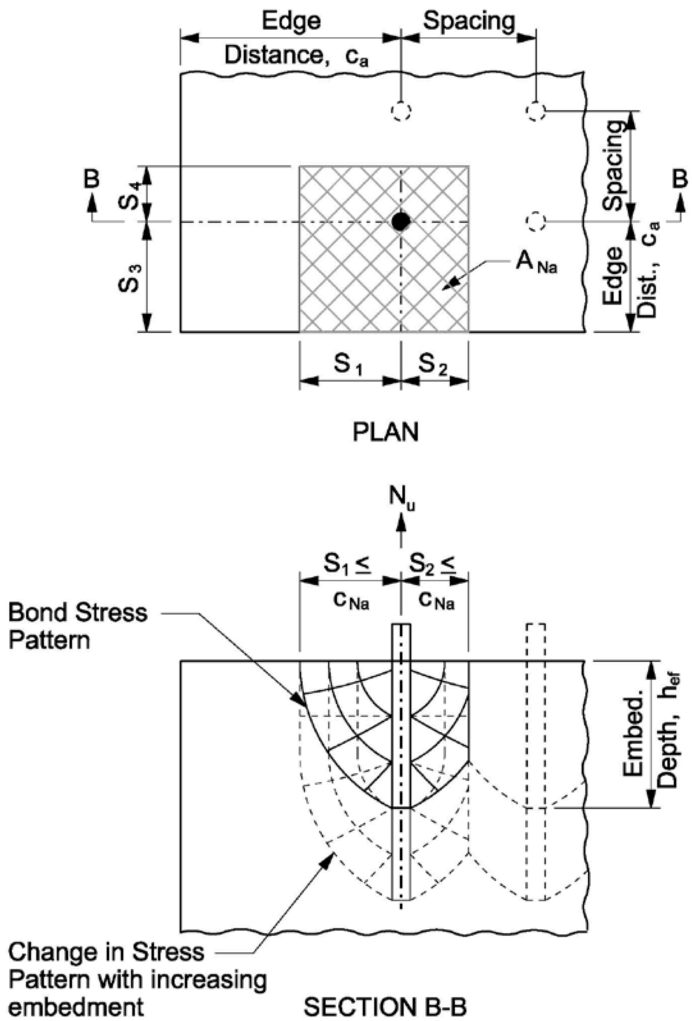

Figure 40.2
Bond Failure of Concrete Adhesive Anchors in Tension

The projected influence area of a single adhesive anchor, A_{Na}, is shown in Figure 40.2. Unlike the concrete breakout area, it is not affected by the embedment depth of the anchor. A_{Na} is limited in each direction by S_i:

S_i = Minimum of:

1. $c_{Na} = 10d_a\sqrt{\frac{\tau_{uncr}}{1100}}$,

2. Half of the spacing to the next anchor in tension, or

3. The edge distance (c_a) (in).

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	820	330	370	280
#5 or 5/8"	820	340	510	290
#6 or 3/4"	820	340	500	290
#7 or 7/8"	820	340	490	290
#8 or 1"	740	340	600	290

Table 40.16-1
Tension Design Table for Concrete Anchors

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for "Concrete Adhesive Anchors". The designer shall determine whether the concrete adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r. For mechanical anchors:

$$N_r = \phi_{ts} N_{sa} \leq \phi_{tc} N_{cb} \leq \phi_{tc} N_{pn}$$

In which:

ϕ_{ts} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.65 for brittle steel as defined in 40.16.1.1
= 0.75 for ductile steel as defined in 40.16.1.1

N_{sa} = Nominal steel strength of anchor in tension, **ACI [17.4.1.2]**
= $A_{se,N} f_{uta}$

$A_{se,N}$ = Effective cross-sectional area of anchor in tension (in^2)

f_{uta} = Specified tensile strength of anchor steel (psi)

$\leq$ $1.9f_{ya}$

$\leq$ 125 ksi

f_{ya} = Specified yield strength of anchor steel (psi)

ϕ_{tc} = Strength reduction factor for anchors in concrete

= 0.65 for anchors without supplementary reinforcement per 40.16.2

= 0.75 for anchors with supplementary reinforcement per 40.16.2

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**

$$= \frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

A_{Nc} = Projected concrete failure area of a single anchor, see Figure 40.1

= $(S_1 + S_2)(S_3 + S_4)$

h_{ef} = Effective embedment depth of anchor per Table 40.16-1. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.

$\psi_{ed,N}$ = Modification factor for tensile strength based on proximity to edges of concrete member, **ACI [17.4.2.5]**

= 1.0 if $c_{a,min} \geq 1.5h_{ef}$

$$= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef}$$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.1 (in)

$\psi_{c,N}$ = Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.2.6]**

= 1.0 when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels

= 1.4 when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

= 1.0 if $c_{a,min} \geq c_{ac}$

= $\frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

c_{ac} = Critical edge distance (in)
= $4.0h_{ef}$

N_b = Concrete breakout strength of a single anchor in tension in uncracked concrete, **ACI [17.4.2.2]**
= $0.538\sqrt{f'_c}(h_{ef})^{1.5}$ (kips)

N_{pn} = Nominal pullout strength of a single anchor in tension, **ACI [17.4.3.1]**
= $\psi_{c,P}N_p$

$\psi_{c,P}$ = Modification factor for pullout strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.3.6]**
= 1.4 where analysis indicates no cracking at service load levels
= 1.0 where analysis indicates cracking at service load levels

N_p = Nominal pullout strength of a single anchor in tension based on the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC193 / ACI 355.2

For adhesive anchors:

$$N_r = \phi_{ts}N_{sa} \leq \phi_{tc}N_{cb} \leq \phi_{tc}N_a$$

In which:

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**
= $\frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

h_{ef} = Effective embedment depth of anchor. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.
≤ $20d_a$ (in)

d_a = Outside diameter of anchor (in)

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

	=	1.0 if $c_{a,min} \geq c_{ac}$
	=	$\dfrac{c_{a,min}}{c_{ac}} \geq \dfrac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$
$c_{a,min}$	=	Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.1 or Figure 40.2 (in)
c_{ac}	=	Critical edge distance (in)
	=	$2.0h_{ef}$
N_a	=	Nominal bond strength of a single anchor in tension, **ACI [17.4.5.1]**
	=	$\dfrac{A_{Na}}{4c_{Na}^{2}}\psi_{ed,Na}\psi_{cp,Na}N_{ba}$
A_{Na}	=	Projected influence area of a single adhesive anchor, see Figure 40.2
	=	$(S_1+S_2)(S_3+S_4)$
$\psi_{ed,Na}$	=	Modification factor for tensile strength of adhesive anchors based on the proximity to edges of concrete member, **ACI [17.4.5.4]**
	=	1.0 if $c_{a,min} \geq c_{Na}$
	=	$0.7+0.3\dfrac{c_{a,min}}{c_{Na}}$ if $c_{a,min} < c_{Na}$
c_{Na}	=	Projected distance from center of anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor
	=	$10d_a\sqrt{\dfrac{\tau_{uncr}}{1100}}$ (in)
τ_{uncr}	=	Characteristic bond stress of adhesive anchor in uncracked concrete, see Table 40.16-1
$\psi_{cp,Na}$	=	Modification factor for pullout strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.5.5]**
	=	1.0 if $c_{a,min} \geq c_{ac}$
	=	$\dfrac{c_{a,min}}{c_{ac}} \geq \dfrac{c_{Na}}{c_{ac}}$ if $c_{a,min} < c_{ac}$
N_{ba}	=	Bond strength in tension of a single adhesive anchor, **ACI [17.4.5.2]**
	=	$\tau_{cr}\pi d_a h_{ef}$

τ_{cr} = Characteristic bond stress of adhesive anchor in cracked concrete, see Table 40.16-1

Note: Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ICC-ES AC308 / ACI 355.4. For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr}.

In addition to the checks listed above for all adhesive anchors, the factored sustained tensile force must be less than or equal to the factored sustained tensile resistance per **ACI [17.3.1.2]**:

$$0.55\phi_{tc}N_{ba} \geq N_{ua,s}$$

40.16.4 Concrete Anchor Shear Capacity

Concrete anchors in shear fail in one of three ways: steel shear rupture, concrete breakout, or concrete pryout. Figure 40.3 shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in Figure 40.3 is limited vertically by H, and in both horizontal directions by S_i:

H = Minimum of:

1. The member depth (h_a) or
2. 1.5 times the edge distance (c_{a1}) (in).

S_i = Minimum of:

1. Half the anchor spacing (S),
2. The perpendicular edge distance (c_{a2}), or
3. 1.5 times the edge distance (c_{a1}) (in).

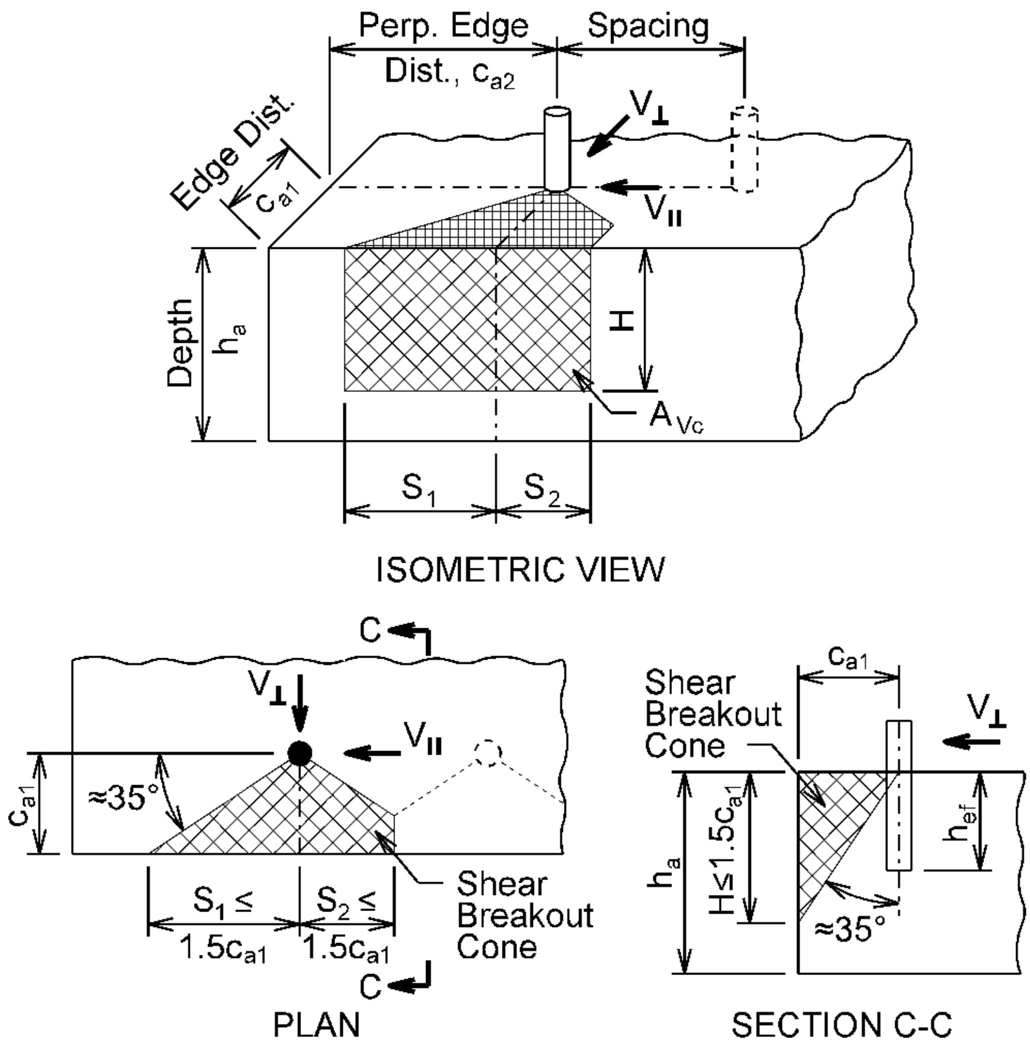

Figure 40.3

Concrete Breakout of Concrete Anchors in Shear

If the shear is applied to more than one row of anchors as shown in Figure 40.4, the shear capacity must be checked for the worst of the three cases. If the row spacing, SP, is at least equal to the distance from the concrete edge to the front anchor, E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the back anchor is checked for the full shear load. If the row spacing, SP, is less than the distance from the concrete edge to the front anchor, E1, then check Case 3. In case 3, the front anchor is checked for the full shear load. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.

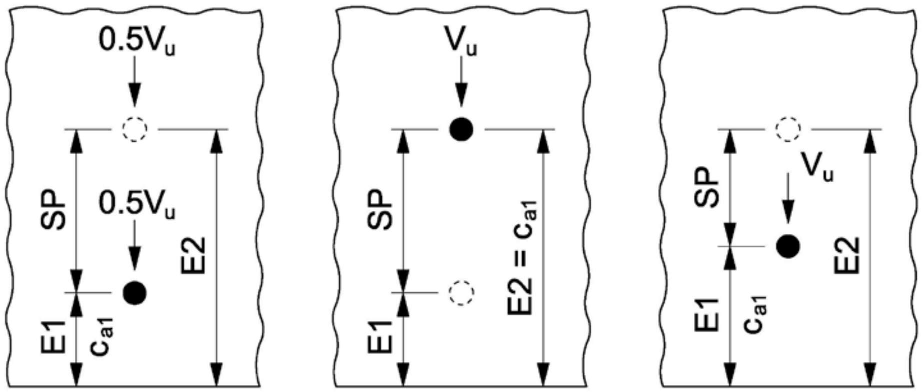

Figure 40.4
Concrete Anchor Shear Force Cases

The factored shear force on each anchor, V_u, must be less than or equal to the factored shear resistance, V_r. For mechanical and adhesive anchors:

$$V_r = \phi_{vs} V_{sa} \leq \phi_{vc} V_{cb} \leq \phi_{vp} V_{cp}$$

In which:

ϕ_{vs} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.60 for brittle steel as defined in 40.16.1.1
= 0.65 for ductile steel as defined in 40.16.1.1

V_{sa} = Nominal steel strength of anchor in shear, **ACI [17.5.1.2]**
= $0.6\, A_{se,V} f_{uta}$

$A_{se,V}$ = Effective cross-sectional area of anchor in shear (in^2)

ϕ_{vc} = Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
= 0.70 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cb} = Nominal concrete breakout strength in shear, **ACI [17.5.2.1]**
= $\dfrac{A_{Vc}}{4.5(c_{a1})^2} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} \Psi_{p,V} V_b$

A_{Vc} = Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.3
= $H(S_1 + S_2)$

c_{a1} = Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.3 and Figure 40.4 (in)

$\psi_{ed,V}$ = Modification factor for shear strength of anchors based on proximity to edges of concrete member, **ACI [17.5.2.6]**
= 1.0 if $c_{a2} \geq 1.5c_{a1}$ (perpendicular shear)
= $0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$ if $c_{a2} < 1.5c_{a1}$ (perpendicular shear)
= 1.0 (parallel shear)

c_{a2} = Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, see Figure 40.3 (in)

$\psi_{c,V}$ = Modification factor for shear strength of anchors based on the presence or absence of cracks in concrete and the presence or absence of supplementary reinforcement, **ACI [17.5.2.7]**
= 1.4 for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels
= 1.0 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels without supplementary reinforcement per 40.16.2 or with edge reinforcement smaller than a No. 4 bar
= 1.2 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge
= 1.4 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at no more than 4 inches

$\psi_{h,V}$ = Modification factor for shear strength of anchors located in concrete members with $h_a < 1.5_{ca1}$, **ACI [17.5.2.8]**
= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

h_a = Concrete member thickness in which anchor is located measured parallel to anchor axis, see Figure 40.3 (in)

$\psi_{p,V}$ = Modification factor for shear strength of anchors based on loading direction, **ACI [17.5]**
= 1.0 for shear perpendicular to the concrete edge, see Figure 40.3
= 2.0 for shear parallel to the concrete edge, see Figure 40.3

V_b = Concrete breakout strength of a single anchor in shear in cracked concrete, per **ACI [17.5.2.2]** , shall be the smaller of:

$$[7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a}]\sqrt{f'_c}(c_{a1})^{1.5} \text{ (lb)}$$

Where:
$l_e = h_{ef} \leq 8d_a$
d_a = Outside diameter of anchor (in)
f'_c = Specified compressive strength of concrete (psi)

and

$$9\sqrt{f'_c}(c_{a1})^{1.5}$$

ϕ_{vp} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cp} = Nominal concrete pryout strength of a single anchor, **ACI [17.5.3.1]**
= $2.0N_{cp}$

<u>Note:</u> The equation above is based on $h_{ef} \geq 2.5$ in. All concrete anchors must meet this requirement.

N_{cp} = Nominal concrete pryout strength of an anchor taken as the lesser of:

mechanical anchors: $\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$

adhesive anchors: $\frac{A_{Na}}{4(c_{Na})^2}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$

and

$\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$

For shear in two directions, check both the parallel and the perpendicular shear capacity. For shear on an anchor near a corner, check the shear capacity for both edges and use the minimum.

40.16.5 Interaction of Tension and Shear

For anchors that are subjected to tension and shear, interaction equations must be checked per **ACI [17.6]**.

If $\frac{V_{ua}}{\phi V_n} \leq 0.2$ for the governing strength in shear, then the full strength in tension is permitted: $\phi N_n \geq N_{ua}$. If $\frac{N_{ua}}{\phi N_n} \leq 0.2$ for the governing strength in tension, then the full strength in shear is permitted: $\phi V_n \geq V_{ua}$. If $\frac{V_{ua}}{\phi V_n} > 0.2$ for the governing strength in shear and $\frac{N_{ua}}{\phi N_n} > 0.2$ for the governing strength in tension, then:

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$

40.16.6 Plan Preparation

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance as determined in 40.16.3.

<u>Typical notes for bridge plans (shown in all capital letters):</u>

Adhesive anchors located in uncracked concrete:

> ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX" IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive anchors located in cracked concrete:

> ADHESIVE ANCHORS X/X-INCH (or No. X BAR). EMBED XX" IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

When using anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item "Adhesive Anchors _-Inch".

For anchors using rebar, the rebar should be listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS Coated Structures".

When adhesive anchors are used as an alternative anchorage the following note should be included in the plans:

> ADHESIVE ANCHORS SHALL CONFORM TO SECTION 502.2.12 OF THE STANDARD SPECIFICATION. *(Note only applicable when the bid item Adhesive Anchor is not used).*

It should be noted that AASHTO is considering adding specifications pertaining to concrete anchors. This chapter will be updated once that information is available.

40.17 Plan Details

1. Excavation for Structures on Overlays

   There is considerable confusion on when to use or not use the bid item "Excavation for Structures" on overlay projects. In order to remove the confusion, the following note is to be added to all overlay projects that only involve removal of the paving block (or less).

   > Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item "(insert applicable bid item)".

   If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay, the "Excavation for Structures" bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements, show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

   Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by current standard of a 0.02 ft/ft cross slope, a cross slope of 0.01 ft/ft or 0.015 ft/ft may be the most desirable.

   The designer should evaluate 3 types of repairs. "Preparation Decks Type 1" is concrete removal to the top of the bar steel. "Preparation Decks Type 2" is concrete removal below the bar steel. "Full Depth Deck Repair" is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of "Full Depth Deck Repair" on the plan sheet.

   Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction, consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

   The Region should provide the designer with a Rehabilitation Structure Survey Report that provides a complete description of the rehabilitation and estimated quantities. Contact the Region for clarifications on the scope of work.

   Additional items:

- Provide deck survey outlining areas of distress (if available). These plans will serve as documentation for future rehabilitations.

- Distressed areas should be representative of the surveyed areas of distress. Actual repairs will likely be larger than the reported values while removing all unsound materials.

- Provide Preparation Deck Type 1 & 2 and Full-Depth Repair estimates for areas of distress.

- Coordinate asphaltic materials with the Region and roadway designers.

See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

40.18 Retrofit of Steel Bridges

Out of plane bending stresses may occur in steel bridges that introduce early fatigue cracks or worse yet brittle fractures. Most, if not all, problems are caused at connections that are either too flexible or too rigid. It is important to recognize the correct condition as retrofitting for the wrong condition will probably make it worse.

40.18.1 Flexible Connections

A connection is too flexible when minor movement can occur in a connection that is designed to be rigid. Examples are transverse or bearing stiffeners fitted to a tension flange, floorbeams attached to the web only and not both flanges.

The solution for stiffeners and transverse connection plates is to shop weld the stiffener to the web and flanges. This becomes a Category C weld but is no different than the terminated web to stiffener weld. If the condition exists in the field, welding is not recommended. Retrofit is to attach a T-section with four bolts to the stiffener and four bolts to the flange. Similar details should be used in attaching floorbeams to the girder.

40.18.2 Rigid Connections

A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.

Other partial constraint details are:

1. Intersecting welds
2. Gap size-allowing local yielding
3. Weld size
4. Partial penetration welds versus fillet welds
5. Touching and intersecting welds

The solution is to create spaces large enough (approximately 1/4” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4” and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

40.19 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

Effective Span Ft-In	T=Slab Thickness Inches	Transverse Bars & Spacing	Longitudinal Bars & Spacing	Longitudinal* Continuity Bars & Spacing
4-0	6.5	#5 @ 8"	#4 @ 8.5"	#5 @ 7.5"
4-3	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-6	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-9	6.5	#5 @ 7"	#4 @ 7.5"	#5 @ 7.5"
5-0	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-3	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-6	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
5-9	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
2-3	7	#4 @ 9"	#4 @ 11"	#5 @ 6.5"
2-6	7	#4 @ 8.5"	#4 @ 11"	#5 @ 6.5"
2-9	7	#4 @ 8"	#4 @ 11"	#5 @ 6.5"
3-0	7	#4 @ 7.5"	#4 @ 11"	#5 @ 6.5"
3-3	7	#4 @ 7"	#4 @ 11"	#5 @ 6.5"
3-6	7	#4 @ 6.5"	#4 @ 11"	#5 @ 6.5"
3-9	7	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6.5"
4-0	7	#4 @ 6"	#4 @ 10"	#5 @ 6.5"
4-3	7	#5 @ 9"	#4 @ 9.5"	#5 @ 7"
4-6	7	#5 @ 8.5"	#4 @ 9"	#5 @ 7"
4-9	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
5-0	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
4-3	7	#5 @ 7.5"	#4 @ 8"	#5 @ 7"
5-6	7	#5 @ 7'	#4 @ 7"	#5 @ 7"
5-9	7	#5 @ 7"	#4 @ 7"	#5 @ 7"
6-0	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-3	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-6	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
6-9	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
7-0	7	#5 @ 6"	#4 @ 6"	#5 @ 6"
4-0	7.5	#4 @ 7"	#4 @ 10.5"	#5 @ 6"
4-3	7.5	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6"
4-6	7.5	#4 @ 6.5"	#4 @ 10"	#5 @ 6"
4-9	7.5	#4 @ 6"	#4 @ 10"	#5 @ 6"
5-0	7.5	#5 @ 9"	#4 @ 9.5"	#5 @ 6"
5-3	7.5	#5 @ 8.5"	#4 @ 9"	#5 @ 6.5"

5-6	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
5-9	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
6-0	7.5	#5 @ 7.5"	#4 @ 8"	#5 @ 6.5"
6-3	7.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 6.5"
6-6	7.5	#5 @ 7"	#4 @ 7.5"	#5 @ 6.5"
6-9	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-0	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-3	7.5	#5 @ 6.5"	#4 @ 6.5"	#5 @ 6.5"
7-6	7.5	#5 @ 6.5"	#5 @ 10"	#5 @ 6.5"
7-9	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-0	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-3	7.5	#5 @ 6"	#5 @ 9.5"	#5 @ 6.5"

Table 40.19-1

Reinforcing Steel for Deck Slabs on Girders for Deck Replacements – HS20 Loading

Max. Allowable Design Stresses: f_c' = 4000 psi, f_y = 60 ksi, Top Steel 2-1/2" Clear, Bottom Steel 1-1/2" Clear, Future Wearing Surface = 20 lbs/ft. Transverse and longitudinal bars shown in table are for one layer only. Place identical steel in both top and bottom layer, except in negative moment region. Use in top layer for slab on steel girders in negative moment region when not designed for negative moment composite action.

40.20 Fiber Reinforced Polymer (FRP)

40.20.1 Introduction

Fiber reinforced polymer (FRP) material is a composite composed of fibers encased in a polymer matrix. The fibers provide tensile strength while the resin protects the fibers and transfers load between them. FRP can be used to repair or to retrofit bridges. Repair is often defined as returning a member to its original condition after damage or deterioration while retrofitting refers to increasing the capacity of a member beyond its original capacity.

For plan preparations, FRP repairs and retrofits are categorized as either structural strengthening or non-structural protection. Contact the Bureau of Structures Design Section for current Special Provisions and for other FRP considerations.

40.20.2 Design Guidelines

While there is no code document for the design of FRP repairs and retrofits, there are two nationally recognized design guidelines: the *Guide Specification for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (14.) hereinafter referred to as the AASHTO FRP Guide, and the *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures ACI 440.2R-08 (15.)* hereinafter referred to as the ACI FRP Guide.

<u>Note: BOS has been evaluating the design methodologies found in the AASHTO FRP Guide and ACI RFP Guide. Noticeable differences between the guides warrants further investigation, with input from industry representation. FRP repairs and retrofits shall be in accordance with the applicable Special Provisions.</u>

40.20.3 Applicability

Not all concrete structures can be retrofitted or repaired using FRP. Most FRP research has been conducted on normal sized members, therefore many of the design equations cannot be used with exceptionally large or deep members. Additionally, members with disturbed regions (D-regions) such as deep beams and corbels are outside of the scope of many design equations.

The structure must have some amount of load carrying capacity prior to the installation of the FRP. Due to the potential for premature debonding, FRP cannot be counted on to carry service loads; it may only be used increase the ultimate capacity of the structure for strength and extreme event load cases. The unrepaired or unretrofitted structure be able to carry the service dead and live loads:

$$R_r \geq \eta_i[(DC + DW) + (LL + IM)]$$

Where:

R_r = factored resistance computed in accordance with AASHTO LRFD Section 5

η_i = load modifier = 1.0

DC = force effects due to components and attachments

DW = force effects due to wear surfaces and utilities

LL = force effects due to live load

IM = force effects due to dynamic load allowance

If capacity is added in flexure to accommodate increased loads, the shear capacity of the member must be checked to ensure that it is still sufficient for the new loading. For non-structural FRP applications, applicability checks may not be required.

40.20.4 Materials

A typical FRP system consists of a primer, fibers, resin, bonding material (either additional resin or an adhesive), and a protective coating. FRP is specified in terms of the types of fiber and resin, the number of layers, the fiber orientation and the geometry. FRP is sold as a system and all materials used should be from the same system.

40.20.4.1 Fibers

The most common types of fiber used for bridge repairs are glass and carbon. Glass fibers are not as stiff or as strong as carbon, but they are much less expensive. Unless there is reason to do otherwise, it is recommended that glass fibers be used for corrosion protection and spall control. Carbon fibers should be used for strengthening and crack control.

Carbon fibers cannot be used where the FRP comes into contact with steel out of concerns for galvanic corrosion due to the highly conductive nature of carbon fibers. For applications where galvanic corrosion is a concern, glass fibers may be used, even in structural applications.

Often, FRP is requested by the region to provide column confinement. The engineer must determine if the requested confinement is true confinement where the FRP puts the column into triaxial compression to increase the capacity and ductility, or if the FRP is confining a patch from spalling off. In the case of true confinement (which is very rare in Wisconsin), carbon fibers should be used and the repair requires structural design. For spall control, glass fibers are acceptable and the repair is considered non-structural.

40.20.4.2 Coatings

After the FRP has been installed and fully cured, a protective coating is applied to the entire system. A protective coating is needed to protect against ultraviolet degradation and can also provide resistance to abrasion, wear, and chemicals. In situations where the FRP is submerged in water, inert protective coatings can help prevent compounds in the FRP from leaching into the water, mitigating environmental impacts.

Protective coatings can be made from different materials depending on the desired coating characteristics. Common coating types include vinyl ester, urethane, epoxy, cementitious, and acrylic. Acrylic coatings are generally the least expensive and easiest to apply, though they may also be less durable. If no coating type is specified, it is likely that the manufacturer will provide an acrylic coating.

For shorter term repairs, acrylic coatings are sufficient, but longer repairs should consider other coating types such as epoxy. Any coating used must be compatible with the FRP system selected by the contractor.

40.20.4.3 Anchors

The bond between the FRP and the concrete is the most critical component of an FRP installation and debonding is the most common FRP failure mode. Certain FRP configurations use anchors to increase the attachment of the FRP and attempt to delay or prevent debonding. These anchors can consist of near surface mounted bars, fiber anchors, additional FRP strips, or mechanical anchors such as bolts. It is permitted to use additional U-wrap strips to anchor flexural FRP, but the use of additional longitudinal strips to anchor shear FRP is prohibited. The use of additional U-wrap strips for flexural anchorage is required in some instances.

Because neither design guide requires anchorage or provides information as to what constitutes anchorage, it is left to the discretion of the designer to determine if anchorage should be used and in what quantities. The use of anchors is highly encouraged, particularly for shear applications and in situations where there is increased potential for debonding such as reentrant corners.

Specifying anchors will add cost to the repair, but these costs may be offset by increased capacity accorded to anchored systems in shear. The additional costs can also be justified if debonding is a concern. If the designer chooses to use anchors, anchors should be shown on plans, but the design of the anchors is left to the manufacturer.

40.20.5 Flexure

Flexural FRP is applied along the tension face of the member, where it acts as additional tension reinforcement. The fibers should be oriented along the length of the member.

40.20.5.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For flexure, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

40.20.5.2 Composite Action

Composite action of the deck slab can be considered when designing flexural FRP repairs for girders, provided that the deck was designed to be composite. If composite action is considered, composite section properties must be computed. These properties should be substituted into the design equations presented in this section. Accounting for composite action will increase the capacity provided by the FRP.

40.20.5.3 Pre-Existing Substrate Strain

Unless all loads are removed from the member receiving FRP (including self-weight), there will be strain present in the concrete when the FRP is applied. This initial or pre-existing substrate strain ε_{bi} is computed through elastic analysis. All loads supported by the member during FRP installation should be considered and cracked section properties should be considered if necessary.

40.20.5.4 Deflection and Crack Control

Conduct standard LRFD serviceability checks for deflection and crack control while accounting for the contribution of the FRP. Because both the FRP and the concrete will be in the elastic zone at service levels, standard elastic analysis can be used to determine stresses and strains. Transformed section analysis can be used to transform the FRP into an equivalent area of concrete for the purposes of analysis. The condition of the member determines if the cracked or uncracked section properties should be used in computations.

40.20.6 Shear

In shear repair/retrofitting applications, the FRP acts essentially as external stirrups. The FRP wrap is applied with the fibers running transverse to the member.

40.20.6.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For shear, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

Additionally, the engineer must ensure that the amount of FRP capacity required does not exceed the maximum allowable shear reinforcement. <u>It is important to note that the FRP capacity listed on the plans will be a factored capacity</u>, while the maximum allowable shear reinforcement check is for an unfactored capacity. Strength reduction factors must be incorporated to make a proper comparison.

If the FRP capacity is close to the maximum allowed, the designer must take care to ensure that a design is feasible. The capacity provided by FRP depends on the number of FRP layers, with each additional layer providing a discrete increase in capacity. There may be a situation where n layers does not provide enough capacity, but n+1 layers provides too much capacity and violates the maximum allowable shear reinforcement criteria. Changes in spacing of the wraps may help decrease the capacity provided by the FRP.

Example problems in shear can be found in the appendices of NCHRP Report 655 (16) and potential shear wrapping configurations can be found in NCHRP Report 678 (17).

40.21 References

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15. ACI. 2008. *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.* Farmington Hills MI: American Concrete Institute, 2008. ACI 440.2R-08.

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45.1 Introduction

Constructed in 1928, the Silver Bridge was an eyebar-chain suspension bridge spanning over the Ohio River between Point Pleasant, West Virginia and Gallipolis, Ohio. On December 15th, 1967 the bridge collapsed, killing 46 people. The resulting investigation revealed that the cause of the collapse was the failure of a single eyebar in a suspension chain. In addition, post-failure analysis showed that the Silver Bridge had been carrying much heavier loads than what it had been originally designed for. At the time of its original design, a typical automobile weighed around 1,500 lbs and the maximum permitted gross weight for a truck was 20,000 lbs. In 1967, those figures had increased to 4,000 lbs and 60,000 lbs respectively.

The Silver Bridge tragedy prompted the bridge engineering community to re-evaluate accepted practice. Clearly, what had been accepted practice was no longer sufficient to guarantee the safety of the travelling public. The Silver Bridge investigation resulted in the development of the National Bridge Inspection Standards (NBIS). These standards require each State Highway Department of Transportation to inspect, prepare reports, and determine load ratings for bridge structures on all public roads. Soon after the development of the NBIS, supporting documents, including the FHWA Bridge *Inspector's Reference Manual* and the AASHTO *Manual for Condition Evaluation of Bridges* were developed to help in implementing these standards.

45.1.1 Purpose of the Load Rating Chapter

The purpose of this chapter is to document Wisconsin Department of Transportation (WisDOT) policy and procedures as they relate to the load rating and load posting of structures in the state of Wisconsin. The development of a load rating may require some degree of engineering judgment. This chapter aims to provide direction on best practice as it relates to these load rating decisions. Guidance is also provided for recommended procedures and required documentation.

45.1.2 Scope of Use

All requirements presented in this chapter are to be followed by WisDOT Bureau of Structures (BOS) staff, as well as any consultants performing load rating or load posting work for WisDOT. Local municipalities and consultants working on their behalf shall also follow the requirements of this chapter.

45.1.3 Governing Standards for Load Rating

The two primary sources for load rating and load posting guidance in Wisconsin are the AASHTO *Manual for Bridge Evaluation (MBE)* and this chapter of the *Wisconsin Bridge Manual*.

AASHTO Manual for Bridge Evaluation (MBE)

In 2011, AASHTO released *The Manual for Bridge Evaluation (MBE)*. The manual replaced the earlier manuals: *The Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (AASHTO LRFR) and Manual for Condition Evaluation of*

Bridges. Although the manual emphasizes the LRFR method, it also provides rating procedures for the Load Factor Rating (LFR) and Allowable Stress Rating (ASR) methodologies. For this reason, it will be the governing manual utilized by WisDOT for load rating structures.

Wisconsin Bridge Manual (WBM), Chapter 45

The Wisconsin Bridge Manual is not an exhaustive resource for load rating and load posting requirements. Unless noted otherwise, this chapter is intended to serve as a supplement to the AASHTO MBE, offering commentary, interpretations, clarification, or additional information as deemed necessary.

Two other commonly utilized references are:

- AASHTO Standard Specification for Highway Bridges, 17th Edition – 2002
- AASHTO LRFD Bridge Design Specifications

See 45.13 for a more complete list of recommended references.

45.1.4 Purpose of Load Rating

Above all else, the primary purpose of a load rating is to ensure that every bridge in the Wisconsin inventory is safe for public use; that it can safely carry legal-weight traffic. The definition of “legal-weight” is discussed in more detail in 45.2.4 and 45.2.5. When the load rating for a bridge decreases beyond a certain threshold – when it can no longer safely carry legal-weight traffic - it may be necessary to restrict heavier loads in order to maintain safety. This is what is referred to as a load posting and is presented in more detail in 45.10.

There are secondary purposes for maintaining load ratings for every structure in the state. Some of these include:

- The Federal Highway Administration (FHWA) requires a current load rating for each bridge as a part of the state National Bridge Inventory (NBI) report.
- Load ratings and load rating analysis files are used for the evaluation of over-weight permit vehicles.
- Decisions on repair and rehabilitation activities are affected by load ratings.
- Decisions on planning for bridge rehabilitation and replacements are affected by load ratings.

45.2 History of Load Rating

This section provides a historical perspective on the load rating process. The intent is to provide a historical context for current load rating and load posting practices in order for load rating engineers to better understand both AASHTO, FHWA, and WisDOT policies.

45.2.1 What is a Load Rating?

A load rating is the relative measure of a structure's capacity to carry live load. As standard practice, FHWA currently requires that two capacity ratings be submitted with the NBI file; the inventory rating and operating rating. The inventory rating is the load level that a structure can safely sustain for an indefinite period. The operating rating is the absolute maximum permissible load level to which a structure may be subjected. As stated above, a load rating is the relative measure of a structure's capacity to carry live load. The logical next question is, "relative to what?" It would be convenient if a simple parameter such as gross vehicle weight could be used to determine a bridge's capacity. However, the actual capacity depends on many factors, such as the gross vehicle weight, the axle configuration, the distribution of loads between the axles, the tire gauge on each axle, etc. It is a generally accepted principle that a bridge that can carry a given load on two axles is capable of carrying the same load (or potentially a larger load) spread over several axles.

In general, FHWA requires that the standard AASHTO HS truck or lane loading be used as the live load when load rating with the Load Factor Rating method (LFR) and the Allowable Stress Rating (ASR) and that the AASHTO HL-93 loading be utilized as the live load when load rating with the Load and Resistance Factor method (LRFR). These standard rating vehicles and rating methodologies are discussed in greater detail in 45.3.6.

45.2.2 Evolution of Design Vehicles

As it is not practical to rate a bridge for the nearly infinite number of axle configurations of trucks on our highways, bridges are rated for standard vehicles that are representative of the actual vehicles in use. As was noted during the investigation of the Silver Bridge collapse (see 45.1), the weight of vehicles travelling over the nation's inventory of bridges has changed dramatically over time. As the size and configuration of vehicles operating on the road has changed, so have the standard design vehicles.

Early bridge design in the United States lacked standardization regarding design live loads. Prior to the widespread presence of automobiles, design live loads were taken as surface loads, intended to represent pedestrian and horse traffic. Documentation in various publications from the early 1900s suggests that 80 psf may have been commonly used. An article in *Engineering News* in 1914 illustrates the opinion that better live load models were necessary, stating, "…these older types of loading are inadequate for purposes of design to take care of modern conditions; they should be replaced by some types of typical motor trucks." A number of live load models were proposed by various entities in the following years, but the first live load that resembled modern day loads was introduced in 1931 in the 1st Edition of the AASHTO Standard Specification for Highway Design. The basic design vehicle in this code was a single unit truck weighing 40,000 lbs. – the H20 design vehicle (See Figure 3.7.6A of the AASHTO Standard Specifications for Highway Bridges, 17th Edition).

As the network of roads and bridges in the United States grew, so did the size and weight of the vehicles operating on them. Recognizing this, the engineering community moved to reflect the changing transportation landscape in the 1944 AASHTO Standard Specification by introducing the HS-20 design vehicle; a tractor-semi trailer combination with three axles, weighing a total of 72,000 lbs. (See Figure 3.7.7A of the AASHTO Standard Specifications for Highway Bridges, 17th Edition) This remains the primary rating vehicle for Load Factor Rating (LFR) and Allowable Stress Rating (ASR). Rating methodologies are discussed further in 45.3.6.

The growth in size and weight of in-service vehicles has continued, and current AASHTO design vehicles are not guaranteed to reflect the actual in-service loading. In the late 1970s and early 1980s, some states moved to using an HS-25 design vehicle in order to more closely approximate an observed increase in the size and weight of truck traffic. Wisconsin adopted an HS-25 design vehicle for a short period of time around 2005 as a precursor to adopting Load and Resistance Factor Design and Rating (LRFD/LRFR).

Discussed in more detail in 45.3.7, LRFD was the next dramatic change in the standard design vehicle. Designated as HL-93, the LRFD design loads include a design vehicle identical to the HS-20, but also include a number of other live load models, including a lane load, a tandem, a double-truck, and a fatigue truck. The HL-93 loading represents the most current design live loads, per AASHTO code. See 17.2.4.2 for a more detailed treatment of the HL-93 loading.

45.2.3 Evolution of Inspection Requirements

In the years following World War II, the United States saw a boom in the construction of roads and bridges. As we're aware today, maintaining accurate, up-to-date documentation on the condition of a bridge is critical to assessing its load carrying capacity; its load rating. However, during this period of expansion, little emphasis was placed on safety inspections or maintenance of in-service bridges. This changed with the Silver Bridge collapse, referenced above. In 1971, the National Bridge Inspection Standards (NBIS) were published, creating national policy regarding inspection procedures, frequency of inspections, qualifications of inspection personnel, inspection reports, and maintenance of state bridge inventories.

While the NBIS represented a dramatic step forward in terms of maintaining safe bridges for the travelling public, the history of bridge design, rating, and inspection is largely reactionary. In the late 1970s, several significant culvert failures prompted an increased emphasis on culverts, eventually resulting in the *Culvert Inspection Manual*, published in 1986. The failure of the Mianus River Bridge in Connecticut in 1983 was a catalyst in the creation of the *Inspection of Fracture Critical Bridge Members*, published in 1986. FHWA published a technical advisory in 1988, *Scour at Bridges*, in response to the collapse of the Schoharie Creek Bridge in New York in 1987 due to scour. Closer to home, the 2000 failure of one of the spans of the Hoan Bridge in Milwaukee, WI brought to national attention to potential danger of highly-constrained connection details. And most recently, the collapse of the I-35W bridge in Minneapolis, MN highlighted the need to more closely inspect and load rate gusset plates. The National Bridge Inspection Standards are under continual review to ensure that the best information is available to engineers who design, load rate, repair, and rehabilitate bridge structures. Discussed in more detail in 45.3.4.3, it is critical that the load rating engineer review the most recent inspection reports and consider the current state of deterioration when load rating a bridge.

45.2.4 Coupling Design with In-Service Loading

As discussed above, design live load vehicles have evolved through the years in an attempt to accurately represent actual in-service traffic. However, until the mid-1950s, there was no legislative connection between the size and weight of in-service traffic and the design capacity of the nation’s bridges. Put more simply, with some local or regional exceptions, it was generally legal to drive any size truck, anywhere. In 1956, this began to change. Congress legislated limits on maximum axle weight (18,000 lbs. on a single axle, 32,000 lbs. for a tandem axle), and gross weight (73,280 lbs.), though there were “grandfather” provisions included. However, even with these limitations, it was still very possible to have a vehicle configuration deemed legal according to the above provisions, but that would induce force effects in excess of the bridge design capacity. Arguably the most significant change in truck size and weight legislation came in 1974 when Congress established the Federal Bridge Formula. The Federal Bridge Formula remains the foundation of truck size and weight legislation today.

45.2.5 Federal Bridge Formula

In the late 1950s, AASHTO conducted an extensive series of field tests to study the effects of truck traffic on pavements and bridges. Based on these tests and an extensive structural analysis effort, the Federal Bridge Formula was developed. The formula is intended to limit the weights of shorter trucks to levels which will limit the overstress in well-maintained bridges designed with HS-20 loading to about 3% and in well-maintained HS-15 bridges to about 30%. While often displayed in table format, the actual formula is as follows.

$$W = 500\{\left[\frac{LN}{N-1}\right] + 12N + 36\}$$

Where: W = the maximum weight in pounds that can be carried on a group of two or more axles to the nearest 500 lbs.

L = the spacing in feet between the outer axles of any two or more axles

N = the number of axles being considered

There are numerous resources readily available to more extensively explain the use of the formula, but it’s important to note that the allowable weight is dependent on the number of axles and the axle spacing. In general, the Federal Bridge Formula is the basis of defining a legal-weight vehicle configuration in Wisconsin. Unless specifically covered via state statute, vehicles that do not conform to the formula must apply for a permit in order to travel over bridges in the Wisconsin. Over-weight truck permitting is discussed further in 45.11. When it is determined that a bridge is not able to safely carry the legal-weight loads, the structure must be load posted. Load postings are discussed in more detail in 45.10.

45.3 Load Rating Process

The following section provides direction on general policies and procedures related to the process for developing a bridge load rating for WisDOT.

45.3.1 Load Rating a New Bridge (New Bridge Construction)

New bridges shall be rated using Load and Resistance Factor Rating (LRFR) methodology. See 45.3.6 for a discussion on rating methodologies.

45.3.1.1 When a Load Rating is Required (New Bridge Construction)

It is mandatory for all new bridges to be load rated. Bridges being analyzed for staged construction shall satisfy the requirements of LRFR for each construction stage. For staged construction, utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by the WisDOT Bureau of Structures Rating Unit.

45.3.2 Load Rating an Existing (In-Service) Bridge

If an existing bridge was designed using LRFD methodology, it shall be rated using LRFR.

If an existing bridge was designed using Load Factor Design (LFD) methodology, it shall be rated using Load Factor Rating (LFR). It is also acceptable to rate using LRFR, but this shall be approved in advance by the WisDOT Bureau of Structures Rating Unit.

If an existing bridge was design using Allowable Stress Design (ASD) methodology, it shall be rated using LFR. It is also acceptable to rate using LRFR, but this shall be approved in advance by the WisDOT Bureau of Structures Rating Unit. There is an exception for bridges with timber or concrete masonry superstructures. For these types only, it is acceptable to utilize Allowable Stress Rating (ASR). See 45.3.6 for a discussion on rating methodologies.

Bridges being analyzed for staged construction during a rehabilitation project shall satisfy the requirements of the appropriate rating methodology (LRFR, LFR, or ASR) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by the WisDOT Bureau of Structures Rating Unit.

Consultants are required to investigate the level of effort required for a given load rating prior to negotiating a contract with WisDOT. This is critical in order to accurately estimate the number of hours required for the load rating. It is also strongly recommended that the rating analysis be performed as early as possible for a rehabilitation project, in the case the ratings are unexpectedly low and the scope of the project requires adjustment in order to improve the ratings.

45.3.2.1 When a Load Rating is Required (Existing In-Service Bridge)

WisDOT policy items:

The load rating effort for rehabilitation projects is intended to be independent of previous ratings. Previous analysis files should be used for information and verification purposes only.

Bridges shall be load rated for any project that results in a change in the loads applied to a structure or to an individual structural element that would typically require a load rating (See 45.3.3 for requirements on what elements should be rated). This requirement includes any of (but is not limited to) the following activities:

- Superstructure replacement
- Deck replacement
- Deck overlays
  - New overlays – concrete, asphalt, or polymer
  - Removal of existing overlays and placement of a new overlay
- Bridge widenings
- Superstructure alterations (re-aligning girders, adding girders, etc.)

(Note: WisDOT recognizes that some of the activities noted above may not result in an appreciable change to the load rating. However, it is WisDOT policy to use these instances as an opportunity for quality control of the load rating for that structure and to verify that the load rating takes into account any current deterioration.)

Bridges shall be load rated if there is noted (inspection reports or otherwise) a significant change in the ability of a member to carry load, i.e. deterioration or distortion.

Bridges require a load rating assessment due to impact damage. This assessment may not necessarily include a re-calculation of the load rating if the damage is deemed to be minimal by a qualified engineer.

45.3.3 What Should be Rated

In general, primary load-carrying members are required to be load rated. Secondary elements may be load rated if there is significant deterioration or if there is question regarding the original design capacity. The load rating engineer is responsible for the decision on load rating secondary elements.

If the load rating engineer, utilizing engineering judgment, determines that certain members or components will not control the rating, then a full analysis of the non-controlling element is not required. Justification for member selection should be clearly stated in the load rating calculations submitted to WisDOT Bureau of Structures. See 45.9 for more information on submittal requirements.

45.3.3.1 Superstructure

- Steel Girder Structures

Primary elements for rating include girders (interior and exterior), floorbeams (if present), and stringers (if present). The concrete deck as it relates to any composite action with the girder (and potentially reinforcing steel in the deck for negative moment applications), is also part of the primary system. While cross frames are considered primary members in a curved girder structure or steel tub girder, these members are not considered to be controlling members, and do not need to be analyzed for load rating purposes. If the inspection report indicates signs of distortion or buckling, the cross frame shall be evaluated and the effects on the adjacent girders considered.

Shiplap joints (if present), and pin-and-hanger joints (if present) also may be considered primary elements. Contact the Bureau of Structures Rating Unit to discuss load ratings for these elements.

Secondary elements include bolted web or flange splices, cross frames and/or diaphragms, stringer-to-floorbeam connections (if present), and floorbeam-to-girder connections (if present).

- Prestressed Concrete Girder Structures

Primary elements for rating include prestressed girders (interior and exterior). The concrete deck (and potentially reinforcing steel in the deck for negative moment applications), as it relates to any composite action with the girder, is also part of the primary system.

Secondary elements include diaphragms.

- Concrete Slab Structures

Primary elements for rating include the structural concrete slab.

Another primary element for rating could include an integral concrete pier cap, if there is no pier cap present. This would take the form of increased transverse reinforcement at the pier, likely combined with a haunched slab design.

- Steel Truss Structures

Primary elements for rating include truss chord members, truss diagonal members, gusset plates connecting truss chord or truss diagonal members, floor beams (if present), and stringers (if present). If any panel points of the truss were designed as braced, bracing members and connections may be considered primary elements.

Secondary elements include splices, stringer-to-floorbeam connections (if present), floorbeam-to-truss connections (if present), lateral bracing, and any gusset plates used to connect secondary elements.

- Timber Girder or Slab Structures

Primary elements for rating include timber girders or timber slab members.

Secondary elements include diaphragms (solid sawn or cross-bracing), stiffener beams, or any tie rods that are present.

- Concrete Box or Channel Structures

Primary elements for rating include concrete box girders.

Secondary elements include diaphragms and shiplap joint connections (if present).

- Additional Elements and Other Structures Types

Transfer girders, straddle bents and/or integral pier caps are considered primary elements. If these elements are present supporting the superstructure to be rated, they are to be included in the load rating.

Other superstructure types should be load rated based on the judgment of the load rating Engineer of Record. The structure types noted below most likely require refined analysis methods to accurately determine the controlling load rating. See 45.3.11 for WisDOT guidance on refined analysis.

  - Steel arch
  - Curved or kinked steel girder
  - Steel tub girder
  - Rigid frame structure (steel or concrete)
  - Steel bascule or vertical lift
  - Cable-stayed or suspension
  - Other more complex structure types that may require efforts beyond typical line girder analysis

As with more typical superstructure types, the load rating engineer should thoroughly review inspection reports when making the decision on what superstructure elements may require a load rating.

45.3.3.2 Substructure

Substructures generally do not control the load rating. Scenarios where substructure element conditions may prompt a load rating include, but are not limited to:

- Collision or impact damage
- Substructure components with significant deterioration, particularly those with a lack of redundancy
- Scour, undermining, or settlement which may affect a footing's bearing capacity or a column's unbraced length

WisDOT policy items:

Reinforced concrete piers are not typically rated. However, if the pier – and particularly the pier cap - has large cracks, significant spalling, or exposed reinforcement that shows deterioration, a more thorough evaluation may be appropriate. Reinforced concrete pier caps exhibiting signs of shear cracks may also warrant further evaluation.

In general, reinforced concrete abutments do not require a load rating. However, if the abutment has large cracks, tipping, displacement, or other movement, a more thorough evaluation may be appropriate.

In either of the cases above, contact the Bureau of Structures Rating Unit to discuss the level of effort required for evaluation.

- Extensive section loss from corrosion or rot. WisDOT recommends reviewing inspection reports and paying particular attention for the following scenarios:
    - Exposed steel pile bents
    - Exposed steel pile abutments
    - Exposed timber pile bents
    - Exposed timber pile abutments
    - Exposed timber pile caps

Based on experience, WisDOT has found the above elements to be particularly susceptible to deterioration, particularly if wet conditions are present. If deterioration is significant, these substructure members may control the rating. In the case of timber piles, calculated ratings may be low, even with little or no deterioration. See 45.7.3 for further discussion on timber piles.

The load rating engineer should thoroughly review inspection reports when making the decision on what substructure elements may require a load rating.

45.3.3.3 Deck

Reinforced concrete decks on redundant, multi-girder bridges are not typically load rated. A load rating would only be required in cases of significant deterioration, damage, or to investigate particularly heavy wheel or axle loads. A deck designed using an antiquated design load (H-10, H-15, etc.) may also warrant a load rating.

Other deck types (timber, filled corrugated steel) generally have lower capacity than reinforced concrete decks. This should be taken under consideration when load rating a structure with one of these deck types. Other deck types may also be more susceptible to damage or deterioration.

It is the responsibility of the load rating engineer to determine if a load rating for the deck is required.

45.3.4 Data Collection

Proper and complete data collection is essential for the accurate load rating of a bridge. It is the responsibility of the load rating engineer to gather all essential data and to assess its reliability. When assumptions are used, they should be noted and justified.

45.3.4.1 Existing Plans

Existing design plans are used to determine original design loads, bridge geometry, member section properties, and member material properties. It is important to review all existing plans; original plans as well as plans for any rehabilitation projects (deck replacements, overlays, etc.). If possible, as-built plans should be consulted as well. These plans reflect any changes made to the design plans during construction. Repair plans that document past repairs to the structure may also be available and should be reviewed, if they exist.

If no plans exist or if existing plans are illegible, field measurements may be required to determine bridge geometries and member section properties. Assumptions may have to be made on material properties. Direction on material assumptions is addressed in 45.5.2.

45.3.4.2 Shop Drawings and Fabrication Plans

Shop drawings and fabrication plans can be an extremely valuable source of information when performing a load rating. Shop drawings and fabrication plans are probably the most accurate documentation of what members and materials were actually used during construction, and may contain information not found in the design plans.

WisDOT has an inventory of shop drawings and fabrication plans, but they do not exist for every existing bridge. If the load rating engineer feels shop drawings and/or fabrication plans are required in order to accurately perform the load rating, contact the Bureau of Structures Rating Unit for assistance.

45.3.4.3 Inspection Reports

When rating an existing bridge, it is critical to review inspection reports, particularly the most recent report. Any notes regarding deterioration, particularly deterioration in primary load-carrying members, should be paid particular attention. It is the responsibility of the load rating engineer to evaluate any recorded deterioration and determine how to properly model that deterioration in a load rating analysis. Reviewing historical inspection reports can offer insight as to the rate of growth of any reported deterioration. Inspection reports can also be used to verify existing overburden.

Inspections of bridges on the State Trunk Highway Network are performed by trained personnel from the Regional maintenance sections utilizing guidelines established in the latest edition of the *WisDOT Structure Inspection Manual*. Engineers from the Bureau of Structures may assist in the inspection of bridges with unique structural problems or when it is suspected that a reduction in load capacity is warranted. To comply with the National Bridge Inspection Standards (NBIS), it is required that all bridges be routinely inspected at intervals not to exceed two years. More frequent inspections are performed for bridges which are posted for load capacity or when it is warranted based on their condition. In addition, special inspections such as underwater diving or fracture critical are performed when applicable. Inspectors enter inspection information into the Highway Structures Information System (HSIS), an on-line bridge management system developed by internally by WisDOT. For more information on HSIS, see 45.3.5. For questions on inspection-related issues, please contact the Bureau of Structures Maintenance Section.

45.3.4.4 Other Records

Other records may exist that can offer additional information or insight into bridge design, construction, or rehabilitation. In some cases, these records may override information found in design plans. It is the responsibility of the load rating engineer to gather all pertinent information and decide how to use that information. Examples of records that may exist include:

- Standard plans – generic design plans that were sometimes used for concrete t-girder structures, concrete slab structures, steel truss structures, and steel through-girder structures.
- Correspondences
- Material test reports
- Mill reports
- Non-destructive test reports
- Photographs
- Repair records
- Historic rating analysis

45.3.5 Highway Structure Information System (HSIS)

The Highway Structure Information System (HSIS) is an on-line database used to store a wide variety of bridge information. Data stored in HSIS is used to create the National Bridge

Inventory (NBI) file that is submitted annually to FHWA. Much of this data can be useful for the load rating engineer when performing a rating. HSIS is also the central source for documents such as plans and maintenance records. Other information, such as design calculations, rating calculations, fabrication drawings, and items mentioned in 45.3.4.4 may also be found in HSIS. For more information on HSIS, see the WisDOT Bureau of Structures web page or contact the Bureau of Structures Bridge Management Unit.

45.3.6 Load Rating Methodologies – Overview

There are two primary methods of load rating bridge structures that are currently utilized by WisDOT. Both methods are detailed in the AASHTO MBE. They are as follows:

- Load and Resistance Factor Rating (LRFR)
- Load Factor Rating (LFR)

Load and Resistance Factor Rating is the most current rating methodology and has been the standard for new bridges in Wisconsin since approximately 2007. LRFR employs the same basic principles as LFR for the load factors, but also utilizes multipliers on the capacity side of the rating equation, called resistance factors, to account for uncertainties in member condition, material properties, etc. This method is covered in 45.3.7, and a detailed description of this method can also be found in **MBE [6A]**.

Load Factor Rating (LFR) has been used since the early 1990s to load rate bridges in Wisconsin. The factor of safety for LFR-based rating comes from assigning multipliers, called load factors, to both dead and live loads. A detailed description of this method can be found in 45.3.8 and also in **MBE [6B]**.

Allowable Stress Rating (ASR) is a third method of load rating structures. ASR was the predominant load rating methodology prior to the implementation of LFR. It is not commonly used for modern load rating, though it is still permitted to be used for select superstructure types (See 45.3.2). The basic philosophy behind this method assigns an appropriate factor of safety to the limiting stress of the material being analyzed. The maximum stress in the member produced by actual loadings is then checked for sufficiency. A more detailed description of this method can be found in 45.3.9 below and also in **MBE [6B]**.

45.3.7 Load and Resistance Factor Rating (LRFR)

The basic rating equation for LRFR, per **MBE [Equation 6A.4.2.1-1]**, is:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

For the Strength Limit States (primary limit state when load rating using LRFR):

$$C = \phi_C \phi_S \phi R_n$$

Where the following lower limit shall apply:

$$\phi_C \phi_S \geq 0.85$$

Where:

RF	=	Rating factor
C	=	Capacity
R_n	=	Nominal member resistance
DC	=	Dead-load effect due to structural components and attachments
DW	=	Dead-load effect due to the wearing surface and utilities
P	=	Permanent loads other than dead loads
LL	=	Live load effects
IM	=	Dynamic load allowance
γ_{DC}	=	LRFR load factor for structural components and attachments
γ_{DW}	=	LRFR load factor for wearing surfaces and utilities
γ_P	=	LRFR load factor for permanent loads other than dead loads = 1.0
γ_{LL}	=	LRFR evaluation live load factor
ϕ_c	=	Condition factor
ϕ_s	=	System factor
ϕ	=	LRFR resistance factor

The LRFR methodology is comprised of three distinct procedures:

- Design Load Rating (first level evaluation) – Used for verification during the design phase, a design load rating is performed on both new and existing structures alike. See 45.3.7.6 for more information.
- Legal Load Rating (second level evaluation) – If required, the legal load rating is used to determine whether or not the bridge in question can safely carry legal-weight traffic; whether or not a load posting is required. See 45.3.7.7 for more information.
- Permit Load Rating (third level evaluation) – The permit load rating is used to determine whether or not over-legal weight vehicles may travel across a bridge. See 45.3.7.8 for more information.

The results of each procedure serve specific uses (as noted above) and also guide the need for further evaluations to verify bridge safety or serviceability. A flow chart outlining this approach is shown in Figure 45.3-1. The procedures are structured to be performed in a sequential manner, as needed, starting with the design load rating. Load rating for AASHTO legal loads is only required when a bridge fails the design load rating (RF < 1.0) at the operating level.

Note that when designing a new structure, it is required that the rating factor be greater than one for the HL-93 vehicle at the inventory level (note also that new designs shall include a dead load allotment for a future wearing surface); therefore, a legal load rating will never be required on a newly designed structure.

Similarly, only bridges that pass the legal load rating at the operating level (RF ≥ 1.0) can be evaluated utilizing the permit load rating procedures. See 45.11 for more information on over-weight permitting.

45.3.7.1 Limit States

The concept of limit states is discussed in detail in the AASHTO LRFD design code (**LRFD [3.4.1]**). The application of limit states to the design of Wisconsin bridges is discussed in 17.2.3.

Service limit states are utilized to limit stresses, deformations, and crack widths under regular service conditions. Satisfying service limits during the design-phase is critical in order for the structure in question to realize its full intended design-life. WisDOT policy regarding load rating using service limit states is as follows:

<u>Steel Superstructures</u>

- The Service II limit state shall be satisfied (inventory rating > 1.0) during design.
- For design or legal load ratings for in-service bridges, the Service II rating shall be checked at the inventory and operating level.
- The Service II limit state should be considered for permit load rating at the discretion of the load rating engineer.

<u>Reinforced Concrete Superstructures</u>

- WisDOT does not consider the Service I limit state during design.
- For design or legal load ratings of new or in-service bridges, the Service I rating is not required.
- The Service I limit state should be considered for permit load rating at the discretion of the load rating engineer.

Prestressed Concrete Superstructures

- The Service III limit state shall be satisfied (inventory rating > 1.0) during the design phase for a new bridge.

- For design load ratings of an in-service bridge, the Service III limit state shall be checked at the inventory level. The Service III limit state should be considered for legal load rating at the discretion of the load rating engineer. The Service III limit state is not required for a permit load rating.

- For design or legal load ratings of new or in-service bridges, the Service I limit state is not required. The Service I limit state should be considered for permit load rating at the discretion of the load rating engineer.

See Table 45.3-1 for live load factors to use for each limit state. Service limit states checks that are considered optional are shaded.

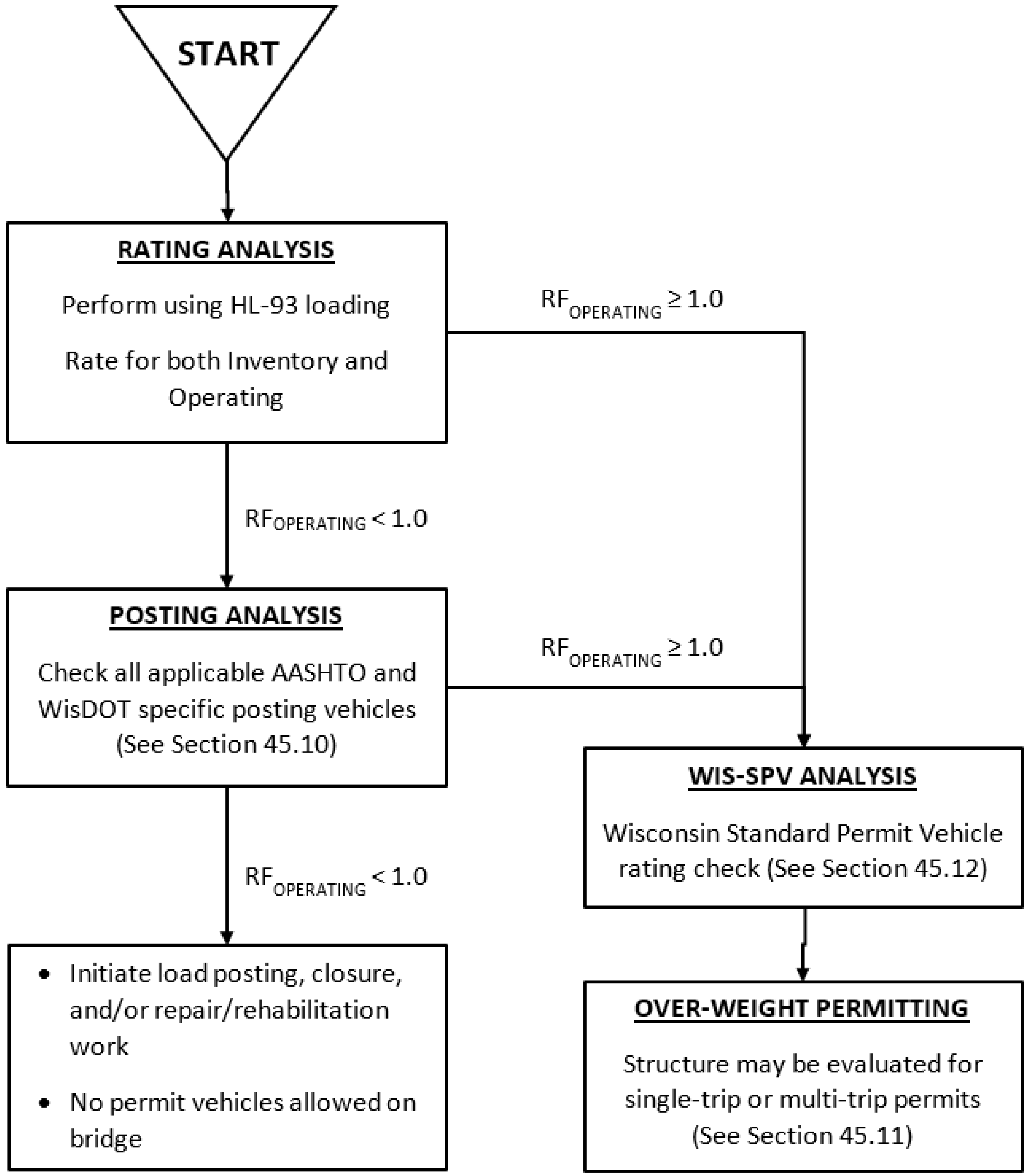


Figure 45.3-1
Load and Resistance Factor Rating Flow Chart

45.3.7.2 Load Factors

The load factors for the Design Load Rating shall be taken as shown in Table 45.3-1. The load factors for the Legal Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-2. The load factors for the Permit Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-3 . Again, note that the shaded values in Table 45.3-1 indicate optional checks that are currently not required by WisDOT.

Bridge Type	Limit State	Dead Load DC	Dead Load DW	Design Load		Legal Load	Permit Load
				Inventory	Operating		
				LL	LL	LL	LL
Steel	Strength I	1.25	1.50	1.75	1.35	Table 45.7-2	Table 45.7-3
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Table 45.7-2	Table 45.7-3
	Service I	1.00	1.00	--	--	--	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Table 45.7-2	Table 45.7-3
	Service III	1.00	1.00	0.80	--	1.00	--
	Service I	1.00	1.00	--	--	--	1.00
Timber	Strength I	1.25	1.50	1.75	1.35	Table 45.7-2	Table 45.7-3

Table 45.3-1
Limit States and Live Load Factors (γ_{LL}) for LRFR

Loading Type	Live Load Factor
AASHTO Legal Vehicles, State Specific Vehicles, and Lane Type Legal Load Models	1.45
Specialized Haul Vehicles (SU4, SU5, SU6, SU7)	1.45

Table 45.3-2
Live Load Factors (γ_{LL}) for Legal Loads in LRFR

Permit Type	Loading Condition	Distribution Factor	Live Load Factor
Annual	Mixed with Normal Traffic	Two or more lanes	1.30
Single Trip	Mixed with Normal Traffic	One Lane	1.20
Single Trip	Escorted with no other vehicles on the bridge	One Lane	1.10

Table 45.3-3
Live Load Factors (γ_{LL}) for Permit Loads in LRFR

45.3.7.3 Resistance Factors

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance. Resistance factors for concrete and steel structures are presented in Section 17.2.6, and resistance factors for timber structures are presented in **MBE [6A.7.3]**.

45.3.7.4 Condition Factor: ϕ_C

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

WisDOT policy items:

Current WisDOT policy is to set the condition factor equal to 1.0.

45.3.7.5 System Factor: ϕ_S

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factor member capacities reduced, and, accordingly, will have lower ratings. The aim of the system factor is to provide reserve capacity for safety of the traveling public. See Table 45.3-4 for WisDOT system factors.

Superstructure Type	ϕ_S
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing ≤ 6.0 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4.0 ft	0.95
All Other Girder and Slab Bridges	1.00
Floorbeam Spacings > 12.0 ft and Non-Continuous Stringers	0.85
Redundant Stringer Subsystems Between Floorbeams	1.00

Table 45.3-4
System Factors for WisDOT

45.3.7.6 Design Load Rating

The design load rating assesses the performance of bridges utilizing the LRFD design loading, producing an inventory and operating rating. Note that when designing a new structure, it is required that the RF > 1.0 at the inventory level. In addition to providing a relative measure of bridge capacity, the design load rating also serves as a screening process to identify bridges that should be load rated for legal loads. If a structure has an operating RF < 1.0, it may not be able to safely carry legal-weight traffic and therefore a legal load rating must be performed. If a structure has a RF ≥ 1.0 at the operating level, proceeding to the legal load rating is not required. However, the load rating engineer is still required to rate the Wisconsin Standard Permit Vehicle (Wis-SPV) as shown in 45.12.

45.3.7.6.1 Design Load Rating Live Load

The LRFD design live load, HL-93, shall be utilized as the rating vehicle(s). The components of the HL-93 loading are described in 17.2.4.2.

45.3.7.7 Legal Load Rating

Bridges that do not satisfy the HL-93 design load rating check (RF < 1.0 at operating level) shall be evaluated for legal loads to determine if legal-weight traffic should be restricted; whether a load posting is required. If the load rating engineer determines that a load posting is required, please notify the Bureau of Structures Rating Unit. For more information on the load posting of bridges, see 45.10.

45.3.7.7.1 Legal Load Rating Live Load

The live loads used for legal load rating calculations are a combination of AASHTO-prescribed vehicles and Wisconsin-specific vehicles. The vehicles to be used for the legal load rating are described in 45.10.

45.3.7.8 Permit Load Rating

Permit load rating is the level of load rating analysis required for all structures when performing the Wisconsin Standard Permit Vehicle (Wis-SPV) design check as illustrated in 45.12. The results of the Wis-SPV analysis are used in the regulation of multi-trip permits. The actual permitting process for State-owned bridges is internal to the WisDOT Bureau of Structures.

Permit load rating is also used for issuance of single trip permits. For each single trip permit, the actual truck configuration is analyzed for every structure it will cross. The single trip permitting process for State-owned bridges is internal to WisDOT Bureau of Structures.

For more information on over-weight truck permitting, see 45.11.

45.3.7.8.1 Permit Load Rating Live Load

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (Figure 45.3-1). Specifics on this analysis can be found in 45.12.

For specific single trip permit applications, the actual truck configuration described in the permit shall be the live load used to analyze all pertinent structures. Permit analysis for State-owned bridges is internal to the WisDOT Bureau of Structures.

WisDOT policy items:

WisDOT interpretation of **MBE [6A.4.5.4.1]** is for spans up to 200'-0", only the permit vehicle shall be considered present in a given lane. For spans 200'-0" in length or greater an additional lane load shall be applied to simulate closely following vehicles. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the permit load effects.

Also note, as stated in the footnote of **MBE [Table 6A.4.5.4.2a-1]**, when using a single-lane LRFD distribution factor, the 1.2 multiple presence factor should be divided out from the distribution factor equations.

45.3.7.9 Load Distribution for Load and Resistance Factor Rating

In general, live load distribution factors should be calculated based on the guidance of the current AASHTO LRFR Standard Design specifications. For WisDOT-specific guidance on the placement and distribution of live loads, see 17.2.7 or 18.4.5.1 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

See also 45.5.1.2 for specific direction on the placement of live loads for rating and posting.

Dead loads shall be distributed as described in 17.2.7 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

45.3.8 Load Factor Rating (LFR)

The basic rating equation for Load Factor Rating can be found in **MBE [Equation 6B.4.1-1]** and is:

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)}$$

Where:

RF	=	Rating factor for the live load carrying capacity
C	=	Capacity of the member
D	=	Dead load effect on the member
L	=	Live load effect on the member
I	=	Impact factor to be used with the live load effect
A_1	=	Factor for dead load
A_2	=	Factor for live load

Unlike LRFR, load factor rating does not have three prescribed levels of rating analysis. However, in practice, the process is similar for both LRFR and LFR.

The first step is to perform a rating analysis to determine inventory and operating ratings. Based on the results of the rating analysis, a posting analysis should be performed when:

- The operating rating factor is less than or equal to 1.2 (HS-24) – Specialized Hauling Vehicles (SHVs) only, see Figure 45.10-2; or
- The operating rating factor is less than or equal to 1.0 (HS-20) for all other posting vehicles.

A posting analysis is performed to determine whether a bridge can safely carry legal-weight traffic. The posting analysis is performed at the operating level. See 45.10 for more information on posting analysis.

Permit analysis is used to determine whether or not over legal-weight vehicles may travel across a bridge. See 45.11 for more information on over-weight vehicle permitting.

A flow chart outlining this approach is shown in Figure 45.3-2. The procedures are structured to be performed in a sequential manner, as needed.

45.3.8.1 Load Factors for Load Factor Rating

See Table 45.3-5 for load factors to be used when rating with the LFR method. The nominal capacity, C, is the same regardless of the rating level desired.

LFR Live Load Factors		
Rating Level	A_1	A_2
Inventory	1.3	2.17
Operating	1.3	1.3

Table 45.3-5
LFR Live Load Factors

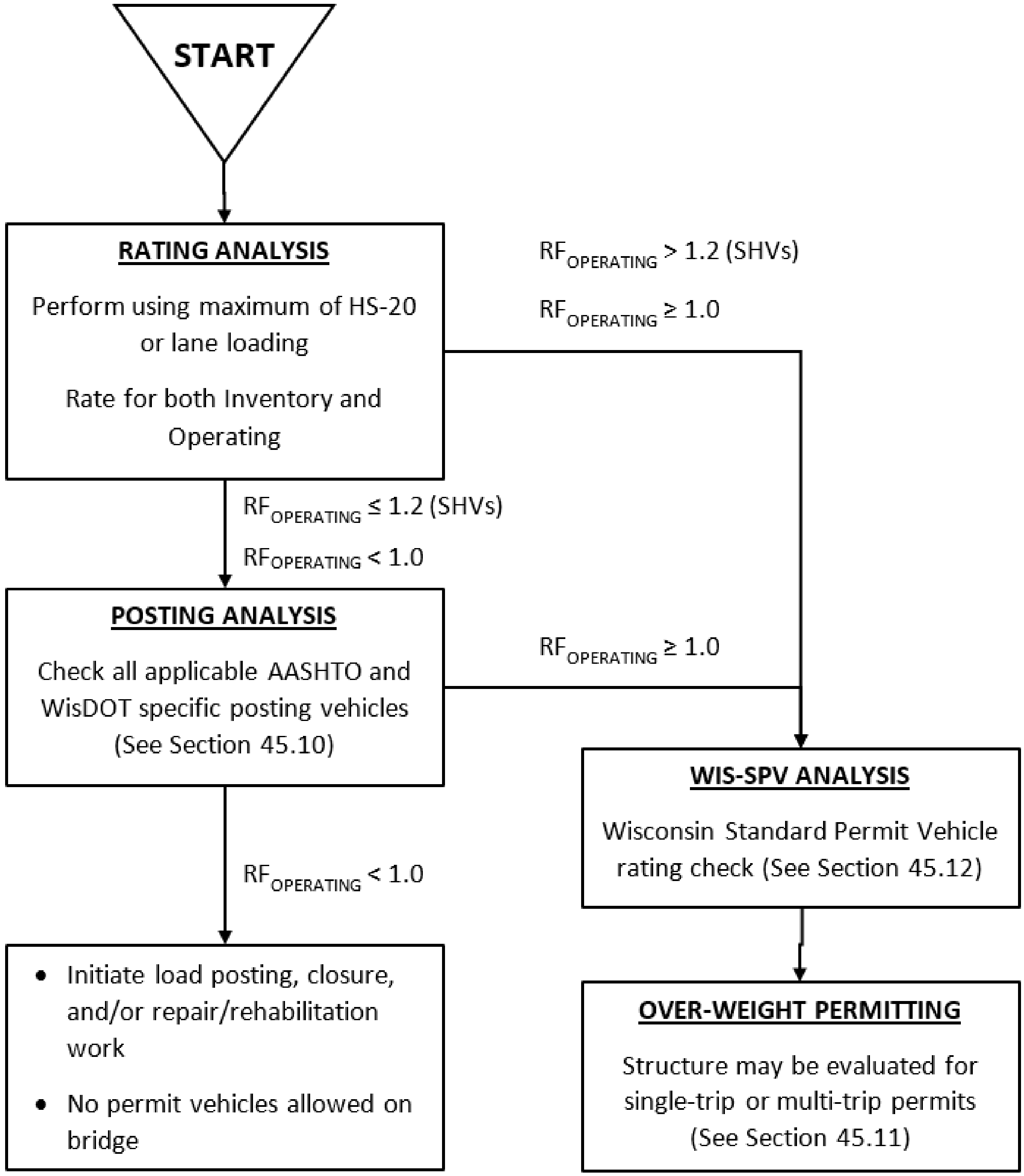


Figure 45.3-2
Load Factor Rating and Allowable Stress Rating Flow Chart

45.3.8.2 Live Loads for Load Factor Rating

Similar to LRFR, there are three potential checks to be made in LFR that are detailed in the flow chart shown in Figure 45.3-2.

- For purposes of calculating the inventory and operating rating of the structure, the live load to be used should be the HS20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3.

- The live load(s) to be used for analysis are a combination of AASHTO-prescribed vehicles and Wisconsin-specific vehicles. For more information on load posting analysis, refer to 45.10.2.

- For conducting the Wisconsin Standard Permit Vehicle analysis, use the loading shown in Figure 45.12-1.

45.3.8.3 Load Distribution for Load Factor Rating

In general, distribution factors should be calculated based on the guidance of the *AASHTO Standard Design Specifications, 17th Edition*.

See 45.5.1.2 for specific direction on the placement of live loads for rating and posting.

Dead loads shall be distributed as described in 17.2.7 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

45.3.9 Allowable Stress Rating (ASR)

The basic rating equation can be found in **MBE [Equation 6B.4.1-1]** and is:

$$RF = \frac{C - D}{L(1 + I)}$$

Where:

RF	=	Rating factor for the live load carrying capacity
C	=	Capacity of the member
D	=	Dead load effect on the member
L	=	Live load effect on the member
I	=	Impact factor to be used with the live load effect

Unlike LRFR, allowable stress rating does not have three prescribed levels of rating analysis. However, in practice, the process is similar for both LRFR and ASR.

The first step is to perform a rating analysis to determine inventory and operating ratings. Based on the results of the rating analysis, a posting analysis should be performed when:

- The operating rating factor is less than or equal to 1.2 (HS-24) – Specialized Hauling Vehicles (SHVs) only, see Figure 45.10-2; or
- The operating rating factor is less than or equal to 1.0 (HS-20) for all other posting vehicles.

A posting analysis is performed to determine whether a bridge can safely carry legal-weight traffic. The posting analysis is performed at the operating level. See 45.10 for more information on posting analysis.

Permit analysis is used to determine whether or not over legal-weight vehicles may travel across a bridge. See 45.11 for more information on over-weight vehicle permitting.

A flow chart outlining this approach is shown in Figure 45.3-2. The procedures are structured to be performed in a sequential manner, as needed.

45.3.9.1 Stress Limits for Allowable Stress Rating

The inventory and operating stress limits used in ASR vary by material. See **MBE [6B]** for more information.

45.3.9.2 Live Loads for Allowable Stress Rating

Similar to LRFR and LFR, there are three potential checks to be made in ASR.

- For purposes of calculating the inventory and operating rating of the structure, the live load to be used should be the HS-20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3.
- The live load(s) to be used for analysis are a combination of AASHTO-prescribed vehicles and Wisconsin-specific vehicles. For more information on load posting analysis, refer to 45.10.2.
- For conducting the Wisconsin Standard Permit Vehicle analysis, use the loading shown in Figure 45.12-1.

45.3.9.3 Load Distribution for Allowable Stress Rating

In general, distribution factors should be calculated based on the guidance of the *AASHTO Standard Design Specifications, 17th Edition*.

See 45.5.1.2 for specific direction on the placement of live loads for rating and posting.

Dead loads shall be distributed as described in 17.2.7 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

45.3.10 Engineering Judgment, Condition-Based Ratings, and Load Testing

Engineering judgment or condition-based ratings alone shall not be used to determine the capacity of a bridge when sufficient structural information is available to perform a calculation-based method of analysis.

Ratings determined by the method of field evaluation and documented engineering judgment may be considered when the capacity cannot be calculated due to one or more of the following reasons:

- No bridge plans available
- Concrete bridges with unknown reinforcement

The engineer shall consider all available information, including:

- Condition of load carrying elements (inspection reports – current and historic)
- Year of construction
- Material properties of members (known or assumed per 45.5.2)
- Type of construction
- Redundancy of load path
- Field measurements
- Comparable structures with known construction details
- Changes since original construction
- Loading (past, present, and future)
- Other information that may contribute to making a more-informed decision

If the engineer of record is considering using a judgment- or inspection-based load rating, a thorough visual observation of the bridge should be conducted, including observing actual traffic patterns for the in-service bridge.

The criteria applied to determine a rating by field evaluation and documented engineering judgment shall be documented via the Load Rating Summary Form (see 45.9) accompanied by any and all related inspection reports, any calculation performed to assist in the rating and assumptions used for those calculations, a written description of the observed traffic patterns for the bridge, relevant correspondences, and any available, relevant photographs of the bridge or bridge condition.

Bridge owners may also consider nondestructive proof load tests in order to establish a safe capacity for bridges in which a load rating cannot be calculated.

WisDOT policy items:

Consult the Bureau of Structures Rating Unit before moving forward with an engineering judgment-based, inspection-based load rating, or with a load testing procedure on either the State or Local system.

45.3.11 Refined Analysis

Methods of refined analysis are discussed in **LRFD [4.6.3]**. These include the use of 2D and 3D finite element modeling of bridge structures, which preclude the use of live load distribution factor equations and instead rely on the relative stiffness of elements in the analytical model for distribution of applied loads. As such, a 2D or 3D model requires the inclusion of elements contributing to the transverse distribution of loads, such as deck and cross frame elements that are otherwise not directly considered in a line girder or strip width analysis. Additional guidance on refined analysis can be found in the AASHTO/NSBA publication "G13.1 Guidelines for Steel Girder Bridge Analysis, 2nd Edition."[19]

WisDOT policy items:

Prior to using refined analysis, consult the Bureau of Structures Rating Unit. Additional documentation is required when performing a refined analysis; see 45.9 for these requirements.

The Bureau of Structures does not require a specific piece of software be used by consultant engineers when performing a refined load rating analysis. See 45.4 for information on load rating computer software.

45.4 Load Rating Computer Software

Though not required, computer software is a common tool used for load rating. WisDOT BOS encourages the use of software for its benefits in increased efficiency and accuracy. However, the load rating engineer must be aware that software is a tool; the engineer maintains responsibility for understanding and verifying any load rating obtained from computer software and should have a full understanding of all underlying assumptions. The load rating engineer is responsible for ensuring that any software used to develop a rating performs that rating in accordance with relevant AASHTO specifications and taking into account specific WisDOT policy noted in this chapter.

45.4.1 Rating Software Utilized by WisDOT

The Bureau of Structures currently uses a mix of software developed in-house and software available commercially. For a list of software currently used by WisDOT for each primary structure type, see the Bureau of Structures website:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/default.aspx

WisDOT does not currently mandate the use of any particular software for load ratings.

45.4.2 Computer Software File Submittal Requirements

When load rating software is used as a tool to derive the load rating for a bridge project (new or rehabilitation), the electronic input file shall be included with the project submittal.

Some superstructure types may require advanced modeling techniques in order to fully and accurately capture the structural response. See 45.3.11 for more information on refined analysis.

See 45.9 (Documentation and Submittals) for more information.

45.5 General Requirements

45.5.1 Loads

45.5.1.1 Material Unit Weights

The following assumptions for material unit weights shall be used when performing a load rating, unless there is project-specific information.

Asphalt	145 pcf
Reinforced Concrete	150 pcf
Soil or Gravel	120 pcf
Steel	490 pcf
Water	62.4 pcf
Timber	50 pcf
½” Thin Epoxy Overlay	5 psf

45.5.1.2 Live Loads

Live loads shall be per 45.3.7 (LRFR), 45.3.8 (LFR), and 45.3.9 (ASR).

WisDOT policy items:

Inventory and operating ratings shall consider the possibility of truck loads on sidewalks. However, posting and permitting analysis need not be calculated with wheel placement on sidewalks.

Lane placement in accordance with AASHTO design specifications may not be consistent with actual usage of a bridge as defined by its striped lanes, and could result in conservative load ratings for bridge types such as trusses, two-girder bridges, ramp structures, arches and bridges with exterior girders governing the ratings via lever rule live load distribution assumptions.

WisDOT policy items:

Upon the approval of the Bureau of Structures Rating Unit, a load rating may be performed by placing truck loads only within the striped lanes. When this alternative is utilized, placement of striped lanes on the bridge shall be field verified and documented in the inspection report, per **MBE [6A.2.3.2]** and **[6B.6.2.2]**.

45.5.1.3 Dead Loads

Dead loads are determined based on the weight and dimensions of the elements in question and shall be distributed as noted in sections above. The following is further guidance offered by WisDOT related to various dead loads.

- The top ½" (or greater if a concrete overlay was placed integral with the deck at the time of pour) of a monolithic concrete deck should be considered a wearing surface. It shall not be considered structural, and thus not used to compute section properties or for composite action.

- For an overlay placed integral with the deck at the time of original construction, the overlay thickness shall be considered a wearing surface. It should not be considered structural, and thus not used to compute section properties or for composite action.

- For a bridge with an existing overlay, only the full remaining thickness of the original deck (original thickness – thickness milled off during overlay process) may be considered structural.

- If the design of a new bridge includes an allowance for a future wearing surface, parapets, sidewalks, or other dead loads, that load shall be excluded during the load rating. A load rating is considered a snapshot of current capacity and should only include loads actually in-place at the time of the rating.

- The weight of the concrete haunch for girder superstructures should be included in the non-composite dead load. The actual average haunch height may be used for load calculations. It is also acceptable to calculate the haunch dead load effect assuming the haunch thickness to vary along the length of the beam, if actual, precise haunch thicknesses are known.

45.5.2 Material Structural Properties

Material properties shall be as stated in AASHTO *MBE* or as stated in this chapter. Often when rating a structure without a complete set of plans, material properties are unknown. The following section can be used as a guideline for the rating engineer when dealing with structures with unknown material properties. If necessary, material testing may be needed to analyze a structure.

45.5.2.1 Reinforcing Steel

The allowable unit stresses and yield strengths for reinforcing steel can be found in Table 45.5-1. When the condition of the steel is unknown, they may be used without reduction. Note that Wisconsin started to use Grade 40 bar steel about 1955-1958; this should be noted on the plans.

Reinforcing Steel Grade	Inventory Allowable (psi)	Operating Allowable (psi)	Minimum Yield Point (psi)
Unknown	18,000	25,000	33,000
Structural Grade	19,800	27,000	36,000
Grade 40 (Intermediate)	20,000	28,000	40,000
Grade 60	24,000	36,000	60,000

Table 45.5-1
Yield Strength of Reinforcing Steel

45.5.2.2 Concrete

The following are the maximum allowable unit stresses in concrete in pounds per square inch (see Table 45.5-2). Note that the "Year Built" column may be used if concrete strength is not available from the structure plans.

Year Built	Inventory Allowable (psi)	Operating Allowable (psi)	Compressive Strength (F'c) (psi)	Modular Ratio
Before 1959	1000	1500	2500	12
1959 and later	1400	1900	3500	10
For all non-prestressed slabs 1975 and later	1600	2400	4000	8
Prestressed girders before 1964 and all prestressed slabs	2000	3000	5000	6
1964 and later for prestressed girders	2400	3000	6000	5

Table 45.5-2
Minimum Compressive Strengths of Concrete

45.5.2.3 Prestressing Steel Strands

Table 45.5-3 contains values for uncoated Seven-Wire Stressed-Relieved and Low Relaxation Strands:

Year Built	Grade	Nominal Diameter of Strand (In)	Nominal Steel Area of Strand (In^2)	Yield Strength (psi)	Breaking Strength (psi)
Prior To 1963	250	$^{7}/_{16}$ (0.438)	0.108	213,000	250,000
Prior To 1963	250	½ (0.500)	0.144	212,500	250,000
1963 To Present	270	½ (0.500)	0.153	229,000	270,000
1973 To Present	270 Low Relaxation	½ (0.500)	0.153	242,500	270,000
1995 to Present	270 Low Relaxation	$^{9}/_{16}$ (0.600)	0.217	242,500	270,000

Table 45.5-3
Tensile Strength of Prestressing Strands

The “Year Built” column is for informational purposes only. The actual diameter of strand and grade should be obtained from the structure plans.

45.5.2.4 Structural Steel

The **MBE [Table 6B.5.2.1-1]** gives allowable stresses for steel based on year of construction or known type of steel. For newer bridges, refer to AASHTO design specifications.

Steel Type		AASHTO Designation	ASTM Designation	Minimum Tensile Strength, Fu (psi)	Minimum Yield Strength, Fy (psi)
Unknown Steel	Built prior to 1905			52,000	26,000
	1905 to 1936			60,000	30,000
	1936 to 1963				33,000
	After 1963				36,000
Carbon Steel		M 94 (1961)	A 7 (1967)	60,000	33,000
Nickel Steel		M 96 (1961)	A 8 (1961)	90,000	55,000
Silicon Steel	up to 1-1/8" thick	M 95 (1961)	A 94	75,000	50,000
	1-1/8" to 2" thick		A 94	72,000	47,000
	2" to 4" thick		A 94 (1966)	70,000	45,000
Low Alloy Steel			A441	75,000	50,000

Table 45.5-4
Minimum Yield Strength Values for Common Steel Types

45.5.2.5 Timber

If plans are available, values and adjustment factors will be taken from the most recent edition of the *National Design Specifications for Wood Construction* (NDS) based on the species and grade of the timber as given on the plans. On older plans that may give the stresses, the stress used for the ratings will be the values from the NDS that correspond with the applicable capacity provisions. If plans are not available, Table 45.5-5 shall be used to estimate the allowable stresses.

For operating ratings, all stresses, in determining capacity, will be multiplied by 1.33.

Bridge Type	Component	Species and Grade	Bending Stress (F_b), psi	Shear Stress (F_v), psi
Longitudinal Nail Laminated Slab Bridges	Slab	Douglas Fir-Larch No. 1 & Btr NDS 2012 Table 4A	1200	180
Longitudinal Glued Laminated Slab Bridges	Slab	20F-V7 NDS 2012 Table 5A	2000	265
Girder-Deck Bridges	Girder, Glu-lam	20F-V7 NDS 2012 Table 5A	2000	265
	Girder, Solid-Sawn	Douglas Fir-Larch Select Structural NDS 2012 Table 4D	1600	170
	Transverse Deck, Glulam	20F-V7 NDS 2012 Table 5A	1600	265
	Transverse Deck, Solid-Sawn	Douglas Fir-Larch No. 1 & Btr NDS 2012 Table 4A	1200	180
Longitudinal Stress-laminated Bridges	Slab, Glu-lam	20F-V7 NDS 2012 Table 5A	2000	265
	Slab, Solid Sawn	Douglas Fir-Larch No. 1 & Btr NDS 2012 Table 4A	1200	180
Substructure Components		**Species and Grade**	**Compression Stress (F_C) psi**	**E_{min} psi**
Timber Piles		Pacific Coast Douglas Fir NDS 2012 Table 6A	1300	690,000

Table 45.5-5

Maximum Allowable Stress for Timber Components

45.5.2.5.1 Timber Adjustment Factors

The following is guidance offered by WisDOT related to timber adjustment factors.

- Load Duration (C_D): Bending, shear, and compression stresses shall be multiplied by 1.15 (traffic load duration).
- Wet Service (C_M): Bending and shear stresses shall be multiplied by the appropriate factor per the footnotes in NDS by assuming that the timber is wet in service. An exception to this is if the rating engineer considers the deck's surface to be impervious,

then C_M shall be 1.0. For large glulam girders covered with deck and wearing surface in good condition such that the girders remain dry, C_M = 1.0.

- Beam Stability (C_L): All girders with decks fastened in the normal manner shall be assumed to have continuous lateral stability and C_L shall be 1.0. If the girders are not prevented from rotating at the points of bearing, or rating engineer determines that there is not continuous lateral support on the compression edge, C_L shall be determined by **NDS [3.3.3]**.

- Size (C_F): Bending stresses for sawn lumber shall be multiplied by the appropriate factor per the footnotes in NDS.

- Volume (C_v): Bending stresses for glued laminated timber shall be multiplied by the appropriate factor per the footnotes in NDS.

- Flat Use (C_{fu}): Bending stresses shall be multiplied by the appropriate factor per NDS, for plank decking loaded on the wide face.

- Repetitive Member (C_r): Bending stresses shall be multiplied by 1.15 on longitudinal nail laminated bridges and on nail laminated decks. For deck planks, 1.15 may be used if they are covered by bituminous surface or perpendicular planks for load distribution and are spaced not more than 24" on center.

- Condition Treatment Factor (C_{pt}): Piling, Bending, Shear, and Compression stresses shall be multiplied by: 1.0 for all douglas fir pile installed prior to 1985, and by 0.9 for all other piles.

- Load Sharing Factor (C_{ls}): This shall be typically be 1.0 for all bents. A higher value may be used per **NDS [6.3.11]** when multiple piles are connected by concrete caps or equivalent force distributing elements so that the pile group deforms together.

- Column Stability (C_P): Compression stresses in bents shall by multiplied by C_p per **NDS [3.7]**. "d" in the formula shall be the minimum measured remaining pile dimension. Unless determined otherwise by the rating engineer, it shall be assumed that all the piles shall have the area and C_p of the worst pile.

The adjusted allowable stress used in ratings shall be the given stress multiplied by all the applicable adjustment factors.

45.6 WisDOT Load Rating Policy and Procedure – Superstructure

This section contains WisDOT policy items or guidance related to the load rating of various types of bridge superstructures.

45.6.1 Prestressed Concrete

For bridges designed to be continuous over interior supports, the negative capacity shall come from the reinforcing steel in the concrete deck. Conservatively, only the top mat of steel deck reinforcing steel should be considered when rating for negative moment. If this assumption results in abnormally low ratings for negative moment, contact the Bureau of Structures Rating Unit for consultation.

Elastic gains in prestressed concrete elements shall be neglected for a conservative approach.

Shear design equations for prestressed concrete bridges have evolved through various revisions of the AASHTO design code. Because of this, prestressed concrete bridges designed during the 1960s and 1970s may not meet current shear capacity requirements. Shear capacity should be calculated based on the most current AASHTO code, either LFR or LRFR. Shear should be considered when determining the controlling ratings for a structure. If shear capacities are determined to be insufficient, the load rating engineer of record should contact the Bureau of Structures Rating Unit for consultation.

If an option is given on the structure plans to use either stress relieved or low relaxation strand, or $^{7}/_{16}$" or $^{1}/_{2}$" diameter strand, consult the shop drawings for the structure to see which option was exercised. If the shop drawings are not available, all possible options should be analyzed and the option which gives the lowest operating rating should be reported.

45.6.1.1 I-Girder

Bridges may have varying girder spacing between spans. A common configuration in Wisconsin with prestressed I-girder superstructures is a four-span bridge with continuous girders in spans 2 & 3 and different (wider) girder spacing in spans 1 & 4. If the chosen analysis program is unable to handle girder spacing varying between spans, several analysis runs may need to be done to capture all potential controlling scenarios.

- In the scenario described above, it seems to have been common practice in the past that when the structure received a new deck, the deck would be poured continuous over all four spans, with negative moment reinforcing in the deck included over the piers. If a full-depth concrete diaphragm is present at the piers, it is acceptable practice to rate the structure as a four-span continuous structure. It is also acceptable to rate the structure as originally constructed; simple exterior spans and two interior spans that are continuous. The decision on how to consider this structure configuration is at the discretion of the rating engineer. All assumptions made should be clearly noted in the calculations and in the load rating summary sheet (See Section 45.9.1).

When the shear failure plane crosses multiple stirrup zones, guidance given in the **MBE [6A.5.8]** should be followed to determine an average shear reinforcement area per unit length existing within the shear failure plane. The shear failure plane is assumed to cross through mid-depth of the section with a 45 degree angle of inclination.

It is common practice to use the average haunch height in order to locate the concrete deck in relation to the top of the girder. It is also acceptable to use the actual, precise haunch thicknesses, if they are known. Absent information on the depth of the haunch, 1 ¼" may be assumed. The area of the haunch may be used in calculating section properties, but it is common practice to conservatively ignore for purpose of section properties (haunch dead load must be taken into account). Appropriate consideration of the haunch is the responsibility of the load rating engineer.

45.6.1.1.1 Variable Girder Spacing (Flare)

Girder spacings may vary over the length of a given girder (flared girder configuration). Some analysis software may allow for a varying distribution factor along the length of the girder. This is the most accurate and thus preferred method for dealing with a flared girder layout.

Alternatively, conservative assumptions may be made regarding the live load distribution and the assigned dead load to the girder being analyzed. The rating engineer is responsible for determining the appropriate assumptions and for ensuring that they produce conservative results. The methods described in **LRFD [C4.6.2.2.1]** are acceptable. All assumptions made shall be clearly noted in the calculations and in the load rating summary sheet (See 45.9.1).

45.6.1.2 Box and Channel Girders

For adjacent prestressed box and channel girders, the concrete topping may be considered structural when rebar extends from the girders up into the concrete topping.

45.6.2 Cast-in-Place Concrete

45.6.2.1 Slab (Flat or Haunched)

When using Load Factor Rating (LFR) or Allowable Stress Rating (ASR) and calculating the single lane load distribution factor for concrete slab bridges, the distribution width, E, shall be taken as 12'-0".

Some concrete slab bridges may have been designed with an integral concrete pier cap. This would take the form of increased transverse reinforcement at the pier, most likely combined with a haunched slab design. It is WisDOT experience that the integral pier cap will very rarely control the load ratings and a specific evaluation is not required. However, if the pier cap shows signs of distress, a more detailed load rating evaluation may be required. <u>Consult the Bureau of Structures Load Rating Unit in these cases.</u>

45.6.3 Steel

Consistent with the WisDOT policy item in 24.6.10, moment redistribution should not be considered as a part of the typical rating procedure for a steel superstructure. Moment redistribution may be considered for special cases (to avoid a load posting, etc.). Contact the Bureau of Structures Rating Unit with any questions on the use of moment redistribution.

Plastic analysis shall be used for steel members as permitted by AASHTO specifications, including (but not limited to) Article 6.12.2 (LRFR) and Articles 10.48.1, 10.53.1.1, and 10.54.2.1 (LFR). Plastic analysis shall not be used for members with significant deterioration. Per code, sections must be properly braced in order to consider plastic capacity. For questions on the use of plastic analysis, contact the Bureau of Structures Rating Unit.

If there are no plans for a bridge with a steel superstructure carrying a concrete deck, it shall be assumed to be non-composite for purposes of load rating unless there is sufficient documentation to prove that it was designed for composite action and that shear studs or angles were used in the construction.

When performing a rating on a bridge with a steel superstructure element (deck girder, floorbeam, or stringer) carrying a concrete deck, the element should be assumed to have full composite action if it was designed for composite action and it has shear studs or angles that are spaced at no more than 2'-0" centers.

Steel girder bridges in Wisconsin have not typically been designed to use the concrete deck as part of a composite system for negative moment. A typical design will show a lack of composite action in the negative moment regions (i.e., no shear studs). However, if design drawings indicate that the concrete deck is composite with the steel girder in negative moment regions, the negative moment steel in the concrete deck shall conservatively consist of only the top mat of steel over the piers.

For steel superstructures, an additional dead load allowance should be made to account for miscellaneous items such as welds, bolts, connection plates, etc., unless these items are all specifically accounted for in the analysis. See 24.4.1.1 for guidance on this additional dead load allowance. Alternatively, the actual weight of all the miscellaneous items can be tabulated and added to the applied dead load.

WisDOT policy items:

When load rating in-service bridges, WisDOT does not consider the overload limitations of Section 10.57 of the AASHTO Standard Specification.

45.6.3.1 Fatigue

For structures originally designed using LRFD, fatigue shall not be part of the rating evaluation.

For structures originally designed using ASD or LFD, fatigue ratings shall not be reported as the controlling rating. However, a fatigue evaluation may be considered for load ratings accompanying a major rehabilitation effort, if fatigue-prone details (category C or lower) are present. Fatigue detail categories are provided in **LRFD Table [6.6.1.2.3-1]**. Contact WisDOT Bureau of Structures Rating Unit on appropriate level of effort for any fatigue evaluation.

45.6.3.2 Rolled I-Girder, Plate Girder, and Box Girder

Application of the lever rule in calculating distribution factors for exterior girders may be overly conservative in some short-span steel bridges with closely spaced girders and slab overhangs. In this case, the live load bending moment for the exterior girder may be determined by applying the fraction of a wheel line determined by multiplying the value of the interior stringers or beams by:

W_e/S, where:

W_e = Top slab width as measured from the outside face of the slab to the midpoint between the exterior and interior stringer or beam. The cantilever dimension of any slab extending beyond the exterior girder shall not exceed S/2, measured from the centerline of the exterior beam.

S = Average stringer spacing in feet.

Alternately, live load distribution for this case may be determined by refined methods of analysis or with consideration of lane stripe placement as described in 45.5.1.2.

It is common practice to use the average haunch height in order to locate the concrete deck in relation to the top of the girder. It is also acceptable to use the actual, precise haunch thicknesses, if they are known. Absent information on the depth of the haunch, 1 ¼" may be assumed. The area of the haunch may be used in calculating section properties, but it is common practice to conservatively ignore for purpose of section properties (haunch dead load must be taken into account). Appropriate consideration of the haunch is the responsibility of the load rating engineer.

45.6.3.2.1 Curvature and/or Kinked Girders

The effects of curvature shall be considered for all curved steel girder structures. For structures meeting the criteria specified in **LRFD [4.6.1.2.4]** or the **Curved Steel Girder Guide Specification [4.2]**, the structure may be analyzed as if it were straight. However, regardless of the degree of curvature, the effects of curvature on flange lateral bending must always be considered in the analysis, either directly through a refined analysis or through an approximate method as detailed in **LRFD [C4.6.1.2.4b]** or the **Curved Steel Girder Guide Specification [4.2.1]**. This is applicable to discretely braced flanges. If a flange is continuously braced (e.g. encased in concrete or anchored to deck by shear connectors) then it need not be considered. In determining the load rating, flange lateral bending stress shall be added to the major axis bending flange stress, utilizing the appropriate equations specified in LRFD. When using the Curved Steel Girder Guide Specification, flange lateral bending stress reduces the allowable flange stress.

45.6.3.2.2 Skew

Load rating of steel structures with discontinuous cross-frames, in conjunction with skews exceeding 20 degrees shall consider flange lateral bending stress, either directly through a refined analysis or using approximate values provided in **LRFD [C6.10.1]**. This requirement only applies to structures with multi-member cross frames (X or K-brace), and full depth diaphragms between girders. Flange lateral bending stress is most critical when the bottom flange is stiffened transversely (discretely braced). For structures with shorter single member diaphragms (e.g. C or MC-shapes) between girders, where the bottom flange is less restrained, the load rating need not consider flange lateral bending stress due to skew.

Flange lateral bending stress, whether due to skew or curvature, is handled the same in a load rating equation. Refer to the flange lateral bending discussion in 45.6.3.2.1 for more information.

45.6.3.2.3 Variable Girder Spacing (Flare)

Girder spacings may vary over the length of a given girder (flared girder configuration). Some analysis software may allow for a varying distribution factor along the length of the girder. This is the most accurate and thus preferred method for dealing with a flared girder layout.

Alternatively, conservative assumptions may be made regarding the live load distribution and the assigned dead load to the girder being analyzed. The rating engineer is responsible for determining the appropriate assumptions and for ensuring that they produce conservative results. The methods described in **LRFD [C4.6.2.2.1]** are acceptable. All assumptions made should be clearly noted in the calculations and in the load rating summary sheet (See 45.9.1).

If the girders are flared such that the ratio of change in girder spacing to span length is greater than or equal to 0.01, then a refined analysis may be required. Consult the Bureau of Structures Rating Unit for structures that meet this criteria.

45.6.3.3 Truss

45.6.3.3.1 Gusset Plates

WisDOT requires gusset plates to be load rated anytime the loads applied to a structure are altered (see 45.3). Gusset plates should also be evaluated with reports of any significant deterioration. Rating procedures shall follow those specified in the AASHTO MBE.

45.6.3.4 Bascule-Type Movable Bridges

Apply twice the normal dynamic impact factor to live loading of the end floorbeam per **AASHTO LRFD Movable Spec [2.4.1.2.4]**. The end floorbeam will likely control the load rating of bascule bridges built before 1980.

45.6.4 Timber

As a material, timber is unique in that material strengths are tied to the load rating methodology used for analysis (typically ASD or LRFR for timber). Because of this and because of the fact that design/rating specifications have changed through the years, the load rating engineer must carefully consider the appropriate material strengths to use for a given member. When referencing historic plans, WisDOT recommends using the plans to determine the type of material (species and grade), but then using contemporary sources (including tables in 45.5.2.5) to determine material strengths and for rating methodology.

Based on experience, WisDOT recommends evaluating timber superstructures for posting vehicles when the rating factor falls below 1.25 instead of the typical 1.0. See 45.10 for more information on load posting.

45.6.4.1 Timber Slab

For longitudinal nail laminated slab bridges, the wheel load shall be distributed to a strip width equal to:

(wheel width) + 2x(deck thickness).

On bridges that are showing lamination slippage, breakage on the underside, or loose stiffener beam connections, the strip width shall be reduced to:

(wheel width) + 1x(deck thickness).

45.7 WisDOT Load Rating Policy and Procedure – Substructure

45.7.1 Timber Pile Abutments and Bents

Any decay or damage will result in the reduction of the load-carrying capacity based on a loss of cross-sectional area (for shear and compression) or in a reduction of the section modulus (for moment). The capacity of damaged timber bents will be based on the remaining cross-sectional area of the pile and the column stability factor (C_p) using “d”, the least remaining dimension of the column. Such reductions will be determined by the rating engineer based on field measurements, when available.

Timber piles with significant deterioration and/or tipping shall be load rated with consideration of lateral earth pressure and redundancy. Piles shall be assumed to be fixed 6 feet below the stream bed or ground line and pinned at their tops.

45.8 WisDOT Load Rating Policy and Procedure – Culverts

45.8.1 Rating New Culverts

Ratings for new bridge-length culverts should be determined based on culvert type. See below for more guidance and see 45.9 for documentation and submittal requirements.

45.8.1.1 New Concrete Box Culverts

Concrete box culverts shall be load rated per AASHTO specifications.

The fill depth in relation to the structure dimensions will determine the live load effect on the structure. For structures that experience little or no live load based on analysis, the ratings reported on plans and in the load rating summary form shall not exceed the ratings noted below:

- Inventory rating factor: 2.0
- Operating rating factor: 3.0

45.8.1.2 New Concrete Pipe Culverts

A concrete pipe culvert system (culvert and fill) shall be designed to carry HL-93 loading. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67

45.8.1.3 New Steel Pipe Culverts

A steel pipe culvert system (culvert and fill) shall be designed to carry HL-93 loading. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67

45.8.2 Rating Existing (In-Service) Culverts

Ratings for existing (in-service) bridge-length culverts shall be determined based on culvert type and the depth of fill over the culvert. See below for more guidance and see 45.9 for documentation and submittal requirements.

45.8.2.1 In-Service Concrete Box Culverts

In-service concrete box culverts with 6'-0" or less of fill may require a load rating. In-service concrete box culverts with more than 6'-0" of fill over the top slab and in good condition based on the most recent inspection report do not require a calculated load rating. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Maximum Vehicle Weight (MVW): 190 kips

WisDOT policy items:

For in-service concrete boxes with less than 6'-0" of fill or with more than 6'-0" of fill, but in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.8.2.2 In-Service Concrete Pipe Culverts

An in-service concrete pipe culvert in good condition does not require a calculated load rating. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Maximum Vehicle Weight (MVW): 190 kips

WisDOT policy items:

For in-service concrete pipe culverts in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.8.2.3 In-Service Steel Pipe Culverts

An in-service steel pipe culvert in good condition does not require a calculated load rating. Ratings shall be reported as:

- Inventory rating factor: 1.0
- Operating rating factor: 1.67
- Maximum Vehicle Weight (MVW): 190 kips

WisDOT policy items:

For in-service steel pipe culverts in poor condition, contact the Bureau of Structures Rating Unit for direction on what is required for a load rating.

45.9 Load Rating Documentation and Submittals

The Bridge Rating and Management Unit is responsible for maintaining information for every structure in the Wisconsin inventory, including load ratings. This information is used throughout the life of the structure to help inform decisions on potential load postings, repairs, rehabilitation, and eventual structure replacement. That being the case, it is critical that WisDOT collect and store complete and accurate documentation regarding load ratings.

45.9.1 Load Rating Calculations

The rating engineer is required to submit load rating calculations. Calculations should be comprehensive and presented in a logical, organized manner. The submitted calculations should include a summary of all assumptions used (if any) to derive the load rating.

45.9.2 Load Rating Summary Forms

After the structure has been load rated, the WisDOT Bridge Load Rating Summary Form shall be completed and utilized as a cover sheet for the load rating calculations (see Figure 45.9-1). This form may be obtained from the Bureau of Structures or is available on the following website:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/plan-submittal.aspx

If required, the Refined Analysis Rating Form (see 45.9.5 and Figure 45.9-2) is available at the same location.

If required, the Culvert Load Rating Summary Form (Figure 45.9-3) is available at the same location.

Instructions for completing the forms are as follows:

Load Rating Summary Form

1. Fill out applicable Bridge Data, Structure Type, and Construction History information using HSIS as reference.

2. Check what rating method and rating vehicle was used to rate the bridge in the spaces provided.

3. Enter the inventory/operating ratings, controlling element, controlling force effect, and live load distribution factor for the rating vehicle.

    a. If the load distribution was determined through refined methods (i.e., finite element analysis), it is not necessary to record the live load distribution factor. Instead, enter “REFINED” in the space provided and use the “Remarks/Recommendations” section to describe the methods used to determine live load distribution.

4. The rating for the Wisconsin Special Permit vehicle (Wis-SPV) is always required and shall be given on the rating sheet for both a multi-lane distribution and a single-lane distribution. Make sure not to include the future wearing surface in these calculations. All reported ratings are based on current conditions and do not reflect future wearing surfaces. Enter the Maximum Vehicle Weight (MVW) for the Wis-SPV analysis, controlling element, controlling force effect, and live load distribution factor.

5. When necessary, posting vehicles shall be analyzed and load postings determined per the requirements of 45.10.

    a. Enter the lowest operating rating in kips for each appropriate vehicle type, along with corresponding controlling element and force effect, as well as live load distribution factor.

6. If a posting vehicle analysis was performed, check the box indicating if a load posting is required or not required. If analysis shows that a load posting is required, specify the level of posting and contact the Bureau of Structures Rating Unit immediately.

7. Enter all additional remarks as required to clarify the load capacity calculations.

8. It is necessary for the responsible engineer to sign and seal the form in the space provided. This is true even for rehabilitation projects with no change to the ratings.

Culvert Load Rating Summary Form

1. Engineered, cast-in-place box culverts should use the Load Rating Summary Form. The Culvert Load Rating Summary Form is intended for other culvert types, including pipe culverts, arch culverts, and precast concrete box culverts.

2. Design overburden depth should be taken from the design calculations/documents.

3. Overburden depth is the current, in-service depth of overburden on the culvert structure.

4. If load ratings are available, they should be recorded. If load ratings are unknown, see 45.8 for direction.

45.9.3 Load Rating on Plans

The plans shall contain the following rating information:

- Inventory Load Rating – The plans shall have either the HS value of the inventory rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. See 6.2.2.3.4 for more information on reporting ratings on plans.

- Operating Load Rating – The plans shall have either the HS value of the operating rating if using LFR or the rating factor for the HL-93 if using LRFR. For LFR ratings, the rating should be rounded down to the nearest whole number. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. See 6.2.2.3.4 for more information.

- Wisconsin Special Permit Vehicle – The plans shall also contain the results of the Wis-SPV analysis utilizing single-lane distribution and assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. The recorded rating for this is the total allowable vehicle weight rounded down to the nearest 10 kips. If the value exceeds 250 kips, limit the plan value to 250 kips. See 6.2.2.3.4 for more information.

45.9.4 Computer Software File Submittals

If analysis software is used to determine the load rating, the software input file shall be provided as a part of the submittal. The name of the analysis software and version should be noted on the Load Rating Summary form in the location provided.

45.9.5 Submittals for Bridges Rated Using Refined Analysis

Additional pages of documentation are required when performing a refined analysis. In addition to the Load Rating Summary Form, also submit the Refined Analysis Rating Form as shown in Figure 45.9-2.

45.9.6 Other Documentation Topics

Structures with Two Different Rating Methods

There may be situations where a given superstructure contains elements that were constructed at different times. In these situations, two different rating methods are used during the design/rating process. For example, a girder replacement or widening. In this case, the new girder(s) would be designed/rated using LRFR, while the existing girders would be rated using LFR. A Load Rating Summary Form shall be submitted for both new & existing structure analysis methods; controlling LRFR rating of the new superstructure components, and controlling LFR rating of the existing superstructure. Both sets of controlling rating values (new & existing) shall be noted on the plan set, as noted in 6.2.2.3.4.

BRIDGE DATA

Bridge Number:		Traffic Count:	
Region:		Traffic Year:	
Owner:		Truck Traffic %:	
Municipality:		Prior Inspection Date:	
Feature On:		Overburden Depth (in):	
Feature Under:		NBI Condition Ratings:	
Design Loading:		Deck: Superstructure: Substructure: Culvert:	

STRUCTURE TYPE

Span #	Material	Configuration	Length (ft)

CONSTRUCTION HISTORY

Year	Work Performed

BRIDGE LOAD RATING SUMMARY

Rating Method:
- ☐ LRFR
- ☐ LFR
- ☐ ASR
- ☐ Field Evaluation / Eng. Judgment

Rating Vehicle: ☐ HL-93 ☐ HS20 ☐ User Defined (Describe in Remarks)

Ratings:	Controlling Element	Controlling Force Effect	LL Distribution Factor*
Inventory:			
Operating:			

* Enter "REFINED" if using a refined analysis. Submit Refined Analysis Rating Form.

Wisconsin Standard Permit Vehicle (Wis-SPV)	MVW (kips)	Controlling Element	Controlling Force Effect	LL Distribution Factor
Single Lane (w/o FWS)				
Multi Lane (w/o FWS)				

Posting Vehicles *	Vehicle GVW (Kips)	Operating Rating (Kips)	Controlling Element	Controlling Force Effect	LL Distribution Factor
Type 3	50				
Type 3S2	72				
Type 3-3	80				
SU4	54				
SU5	62				
SU6	69.5				
SU7	77.5				
PUP	98				
Semi	98				

Load Posting: ☐ Not Required ☐ Required:

* Posting Vehicle Analysis (when required per Wisconsin Bridge Manual, Chapter 45)

Load Rating Engineer

Name:

Date:

PE Stamp Here

Computer Software Used (Name/Version):

Additional Remarks:

Figure 45.9-1

Bridge Load Rating Summary Form

In Addition to this form, submit electronic analysis files (eg. .MDX, .bdb)

ANALYSIS FILE SUMMARY (FILL OUT FOR EACH ANALYSIS FILE SUBMITTED)

Analysis Type:	☐ Grid/Grillage ☐ Plate & Ecc. Beam ☐ 3D FEM ☐ Other *(describe below)*
Analysis Program:	☐ MDX ☐ AASHTOWare ☐ CSI Bridge ☐ LARSA ☐ Other
Program Version:	
File Name:	
File Description:	Describe the purpose of the file. Example: This file is used for the Wis-SPV rating using single lane distribution.
Analysis Assumptions:	Highlight key assumptions in modeling. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Example of things to include: a description of the finite element model, simplifications made to model, exceptions to original design plans, loads applied, how loads are applied (e.g. equally distributed to all girders), support conditions, composite/non-composite sections.
Summary of Results:	Summarize results. (This section may be omitted if submitting MDX or AASHTOWare analysis files. This section may also be omitted if submitting separate document containing analysis assumptions and results). Provide table of results for service load reactions, moment, shear, and/or stress output for members at 10th points (minimum) for the appropriate load cases. Provide a table of capacities at each 10th point, such that load ratings can be directly computed with appropriate load and/or resistance and impact factors. Provide example or typical calculations.

Figure 45.9-2

Refined Analysis Rating Form

For concrete box culverts, use Bridge Load Rating Summary form. All other bridge-length culverts shall use this form.

CULVERT DATA

Bridge Number:		**Traffic Count:**	
Owner:		**Traffic Year:**	
Municipality:		**Truck Traffic %:**	
Feature On:		**Prior Inspection Date**:**	
Design Loading*:		**Overburden Depth (in)**:**	
Design Overburden Depth (in)*:		**NBI Culvert Condition Rating**:**	
**For new culverts; if known for in-service culverts*		***For in-service culverts only*	

STRUCTURE TYPE

Span #	Material	Configuration	Length (ft)

CONSTRUCTION HISTORY

Year	Work Performed

CULVERT LOAD RATING SUMMARY

Refer to Section 45.8 of the Wisconsin Bridge Manual for instructions on reporting load ratings.

Rating Method: ☐ LRFR ☐ LFR ☐ ASR ☐ Field Evaluation / Eng. Judgment	**Rating Vehicle:** ☐ HL-93 ☐ HS20 ☐ User Defined (Describe in Remarks)	
	RATINGS:	
	Inventory:	
	Operating:	
	MVW (Wisconsin SPV):	kips
	Load Posting:	☐ Not Required ☐ Required (enter posting weight):

Additional Remarks:	**Design or Load Rating Engineer**
	Name:
	Date:
	PE Stamp Here

Figure 45.9-3

Culvert Load Rating Summary Form

45.10 Load Postings

45.10.1 Overview

Legal-weight for vehicles travelling over bridges is determined by state-specific statutes, which are based in part on the Federal Bridge Formula. The Federal Bridge Formula is discussed in 45.2.5. When a bridge does not have the capacity to carry legal-weight traffic, more stringent load limits are placed on the bridge – a load posting. Currently in Wisconsin, load postings are based on gross vehicle weight; there is no additional consideration for number of axles or axle spacing. Load posting signage is discussed further in 45.10.4.

In order to remain open to traffic, a bridge should be capable of carrying a minimum gross live load weight of three tons at the Operating level. Bridges not capable of carrying a minimum gross live load weight of three tons at the Operating level <u>must</u> be closed. As stated in the **MBE [6A.8.1]** and **[6B.7.1]**, when deciding whether to close or post a bridge, the Owner should consider the character of traffic, the volume of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting.

The owner of a bridge has the responsibility and authority to load post a bridge as required. The State Bridge Maintenance Engineer has the authority to post a bridge and must issue the approval to post any State bridge.

<u>WisDOT policy items:</u>

Consult the Bureau of Structures Rating Unit as soon as possible with any analysis that results in a load posting for any structure on the State or Local system.

45.10.2 Load Posting Live Loads

The live loads to be used in the rating formula for posting considerations are any of the three typical AASHTO Commercial Vehicles (Type 3, Type 3S2, Type 3-3) shown in Figure 45.10-1, any of the four AASHTO Specialized Hauling Vehicles (SHVs - SU4, SU5, SU6, SU7) shown in Figure 45.10-2, the WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3, and the Wisconsin Standard Permit Vehicle shown in Figure 45.12-1.

The AASHTO Commercial Vehicles and Specialized Hauling Vehicles are modeled on actual in-service vehicle configurations. These vehicles comply with the provisions of the Federal Bridge Formula and can thus operate freely without permit; they are legal weight/configuration.

The WisDOT Specialized Annual Permit Vehicles are Wisconsin-specific vehicles. They represent vehicle configurations made legal in Wisconsin through the legislative process and current Wisconsin state statutes.

The Wisconsin Standard Permit Vehicle (Wis-SPV) is a configuration used internally by WisDOT to assist in the regulation of multi-trip (annual) permits. Multi-trip permits and the Wis-SPV are discussed in more detail in 45.11.2 and 45.12.

As stated in **MBE [6A.4.4.2.1a]**, for spans up to 200’, only the vehicle shall be considered present in the lane for positive moments. It is unnecessary to place more than one vehicle in a lane for spans up to 200’ because the load factors provided have been modeled for this possibility. For spans 200’ in length or greater, the AASHTO Type 3-3 truck multiplied by 0.75 shall be analyzed combined with a lane load as shown in Figure 45.10-4. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the vehicle load effects.

Also, for negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 trucks multiplied by 0.75 shall be used. The trucks should be heading in the same direction and should be separated by 30 feet as shown in Figure 45.10-4. There are no span length limitations for this negative moment requirement.

When the lane-type load model (see Figure 45.10-4) governs the load rating, the equivalent truck weight for use in calculating a safe load capacity for the bridge shall be taken as 80 kips as is specified in **MBE [6A.4.4.4]**.

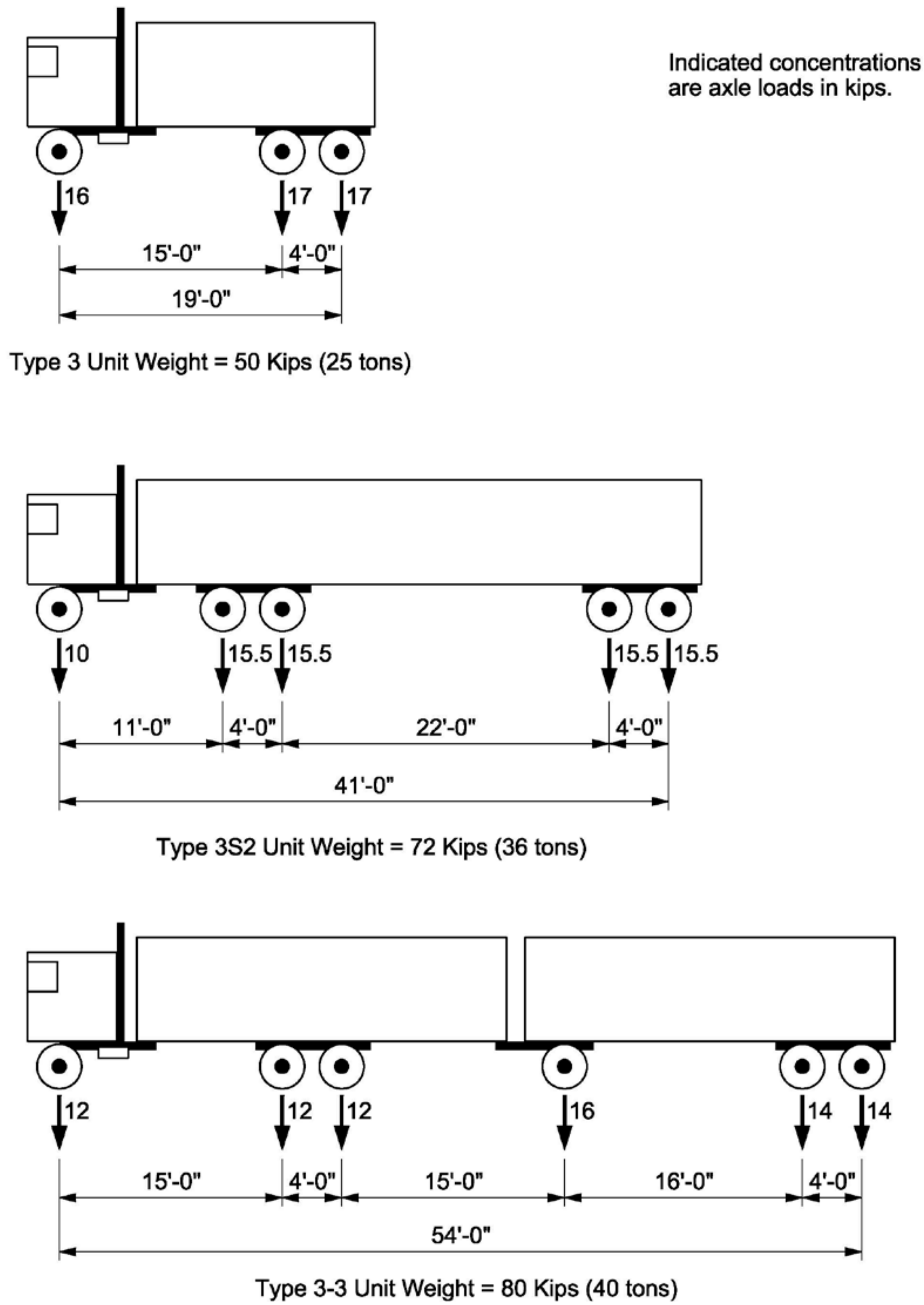


Figure 45.10-1
AASHTO Commercial Vehicles

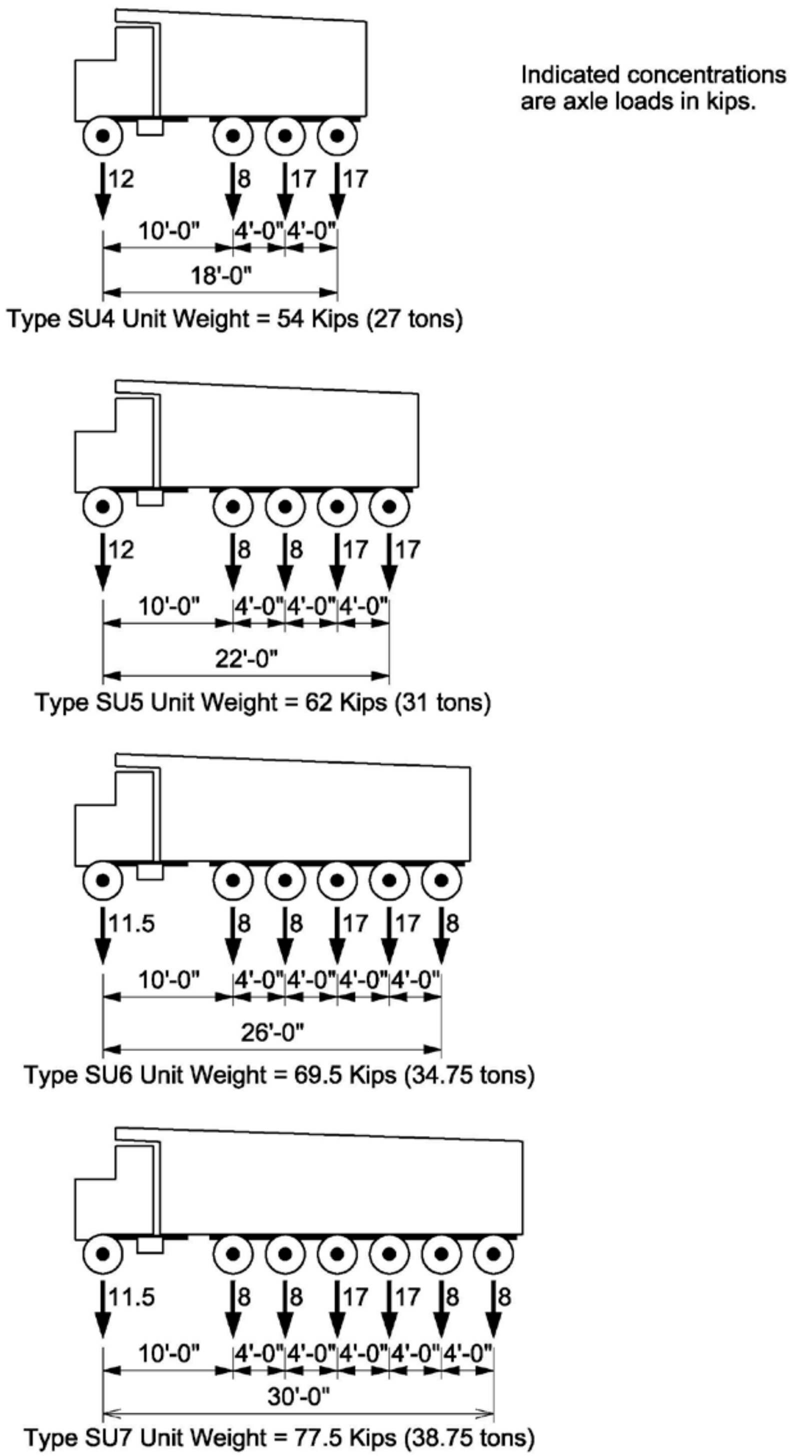


Figure 45.10-2
AASHTO Specialized Hauling Vehicles (SHVs)

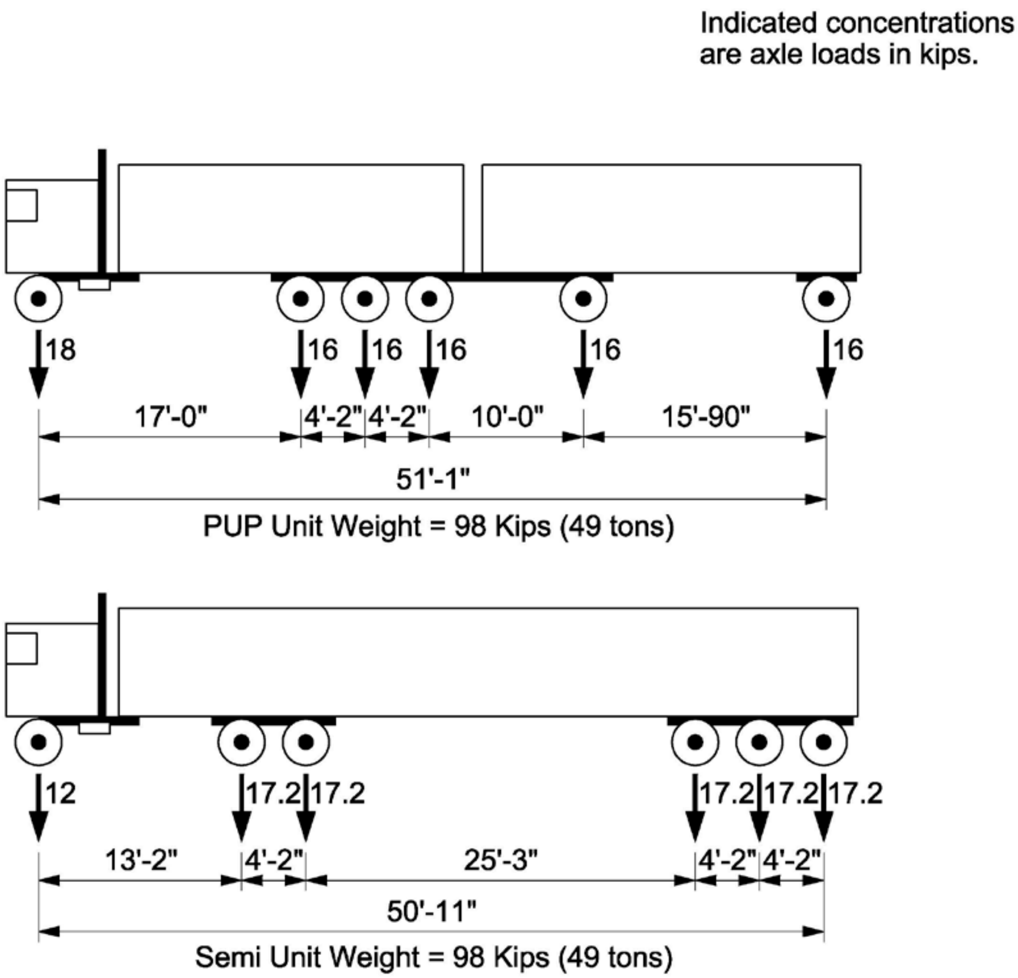
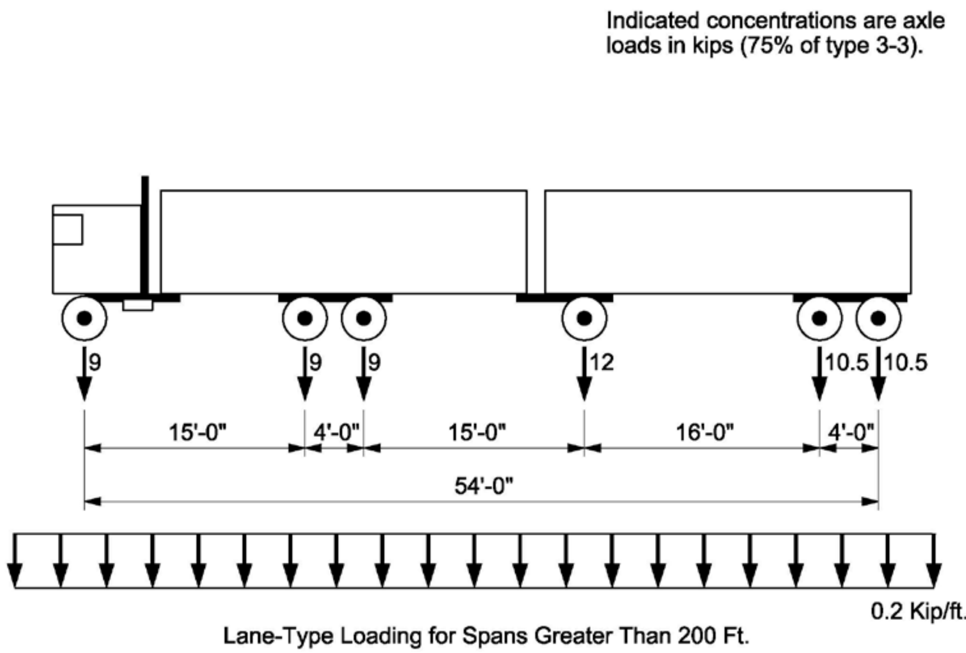


Figure 45.10-3
WisDOT Specialized Annual Permit Vehicles



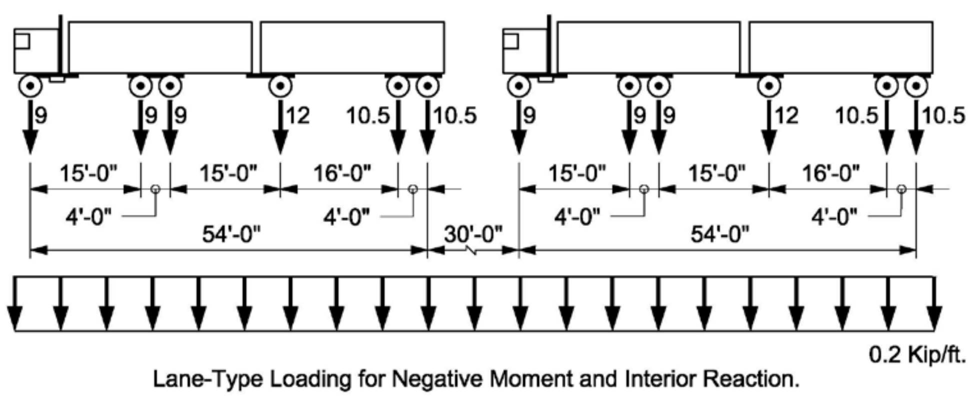


Figure 45.10-4

Lane Type Legal Load Models

45.10.3 Load Posting Analysis

All posting vehicles shall be analyzed at the operating level. A load posting analysis is required when the calculated rating factor at operating level for a bridge is:

- Less than 1.0 for LRFR methodology.
- Less than 1.0 for LFR/ASR methodology; or
- Less than or equal to 1.2 for LFR/ASR methodology (SHV analysis only)
- Less than 1.25 for analysis of timber longitudinal slab superstructures

A load posting analysis is very similar to a load rating analysis, except the posting live loads noted in 45.10.2 are used instead of typical LFR or LRFR live loading.

If the calculated rating factor at operating is less than 1.0 for a given load posting vehicle, then the bridge shall be posted, with the exception of the Wis-SPV. For State Trunk Highway Bridges, current WisDOT policy is to post structures with a Wisconsin Standard Permit Vehicle (Wis-SPV) rating of 120 kips or less. If the RF ≥ 1.0 for a given vehicle at the operating level, then a posting is not required for that particular vehicle.

A bridge is posted for the lowest restricted weight limit of any of the standard posting vehicles. To calculate the capacity, in tons, on a bridge for a given posting vehicle utilizing LFR, multiply the rating factor by the gross vehicle weight in tons. To calculate the posting load for a bridge analyzed with LRFR, refer to 45.10.3.2.

45.10.3.1 Limit States for Load Posting Analysis

For LFR methodology, load posting analysis should consider strength-based limit states only.

For LRFR methodology, load posting analysis should consider strength-based limit states, but also some service-based limit states, per Table 45.3-1.

45.10.3.2 Legal Load Rating Load Posting Equation (LRFR)

When using the LRFR method and the operating rating factor (RF) calculated for each legal truck described above is greater than 1.0, the bridge does not need to be posted. When for any legal truck the RF is between 0.3 and 1.0, then the following equation should be used to establish the safe posting load for that vehicle (see **MBE [Equation 6A8.3-1]**):

$$\text{Posting} = \frac{W}{0.7}\left[(RF) - 0.3\right]$$

Where:

RF = Legal load rating factor

W = Weight of the rating vehicle

When the rating factor for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the bridge. If necessary, the structure may need to be closed until it can be repaired, strengthened, or replaced. This formula is only valid for LRFR load posting calculations.

45.10.3.3 Distribution Factors for Load Posting Analysis

WisDOT policy items:

The AASHTO Commercial Vehicles and Specialized Hauling Vehicles shall be analyzed using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

The WisDOT Specialized Annual Permit Vehicles shown in Figure 45.10-3 shall be analyzed using a single-lane distribution factor, regardless of bridge width.

The Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed for load postings using a multi-lane distribution factor for bridge widths 18'-0" or larger. Single lane distribution factors are used for bridge widths less than 18'-0".

45.10.4 Load Posting Signage

Current WisDOT policy is to post State bridges for a single gross weight, in tons. Bridges that cannot carry the maximum weight for the vehicles described in 45.10.2 at the operating level are posted with the standard sign shown in Figure 45.10-5. This sign shows the bridge capacity for the governing load posting vehicle, in tons. The sign should conform to the requirements of the *Wisconsin Manual for Uniform Traffic Control Devices (WMUTCD)*.

In the past, local bridges were occasionally posted with the signs shown in Figure 45.10-6 using the H20, Type 3 and Type 3S2 vehicles. The H20 represented the two-axle vehicle, the Type 3 represented the three-axle vehicle and the Type 3S2 represented the combination vehicle. This practice is not encouraged by WisDOT and is generally not allowed for State-owned structures, except with permission from the State Bridge Maintenance Engineer.

WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

Figure 45.10-5
Standard Signs Used for Posting Bridges

WEIGHT LIMIT 2 AXLE VEHICLES 15 TONS 3 AXLE VEHICLES 20 TONS COMBINATION VEHICLES 30 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS

Figure 45.10-6
Historic Load Posting Signs

45.11 Over-Weight Truck Permitting

45.11.1 Overview

Size and weight provisions for vehicles using the Wisconsin network of roads and bridges are specified in the Wisconsin Statutes, Chapter 348: Vehicles – Size, Weight and Load. Weight limits for legal-weight traffic and over-weight permit requirements are defined in detail in this chapter. The webpage for Chapter 348 is shown below.

http://docs.legis.wisconsin.gov/statutes/statutes/348

Over-weight permit requests are processed by the WisDOT Oversize Overweight (OSOW) Permit Unit in the Bureau of Highway Maintenance. The permit unit collaborates with the WisDOT Bureau of Structures Rating Unit to ensure that permit vehicles are safely routed on the Wisconsin inventory of bridges.

While the Wisconsin Statutes contain several industry-specific size and weight annual permits, in general, there are two permit types in Wisconsin: multi-trip (annual) permits and single-trip permits.

45.11.2 Multi-Trip (Annual) Permits

Multi-trip permits are granted for non-divisible loads such as machines, self-propelled vehicles, mobile homes, etc. They typically allow unlimited trips and are available for a range of three months to one year. The permit vehicle may mix with typical traffic and move at normal speeds. Multi-trip permits are required to adhere to road and bridge load postings and are subject to additional restrictions based on restricted bridge lists supplied by the WisDOT Bureau of Structures Rating Unit and published by the WisDOT OSOW Permit Unit. The restricted bridge lists are developed based on the analysis of the Wisconsin Standard Permit Vehicle (Wis-SPV). For more information on the Wis-SPV and required analysis, see 45.12. The carrier is responsible for their own routing, and are required to avoid these restrictions and load postings.

Vehicles applying for a multi-trip permit are limited to 170,000 pounds gross vehicle weight, plus additional restrictions on maximum length, width, height, and axle weights. Please refer to the WisDOT Oversize Overweight (OSOW) Permits website or the Wisconsin Statues (link above) for more information.

http://www.dot.wisconsin.gov/business/carriers/osowgeneral.htm

45.11.3 Single Trip Permits

Non-divisible loads which exceed the annual permit restrictions may be moved by the issuance of a single trip permit. When a single trip permit is issued, the applicant is required to indicate on the permit the origin and destination of the trip and the specific route that is to be used. A separate permit is required for access to local roads. Each single trip permit vehicle is individually analyzed by WisDOT for all state-owned structures that it encounters on the designated permit route.

Live load distribution for single trip permit vehicles is based on single lane distribution. This is used because these permit loads are infrequent and are likely the only heavy loads on the structure during the crossing. The analysis is performed at the operating level.

At the discretion of the engineer evaluating the single trip permit, the dynamic load allowance (or impact for LFR) may be neglected provided that the maximum vehicle speed can be reduced to 5 MPH prior to crossing the bridge and for the duration of the crossing.

In some cases, the truck may be escorted across the bridge with no other vehicles allowed on the bridge during the crossing. If this is the case, then the live load factor (LRFR analysis) can be reduced from 1.20 to 1.10 as shown in Table 45.3-3. It is recommended that the truck be centered on the bridge if it is being escorted with no other vehicles allowed on the bridge during the crossing.

Vehicles with non-standard axle gauges may also receive special consideration. This may be achieved by performing a more-rigorous analysis of a given bridge that takes into account the specific load configuration of the permit vehicle in question instead of using standard distribution factors that are based on standard-gauge axles. Alternatively, modifications may be made to the standard distribution factor in order to more accurately reflect how the load of the permit vehicle is transferred to the bridge superstructure. How non-standard gauge axles are evaluated is at the discretion of the engineer evaluating the permit.

45.12 Wisconsin Standard Permit Vehicle (Wis-SPV)

45.12.1 Background

The Wis-SPV configuration is shown in Figure 45.12-1. It is an 8-axle, 190,000lbs vehicle. It was developed through a Wisconsin research project that investigated the history of multi-trip permit configurations operating in Wisconsin. The Wis-SPV was designed to completely envelope the force effects of all multi-trip permit vehicles operating in Wisconsin and is used internally to help regulate multi-trip permits.

45.12.2 Analysis

- New Bridge Construction

For any new bridge design, the Wis-SPV shall be analyzed. The Wis-SPV shall be evaluated at the operating level. When performing this design check for the Wis-SPV, the vehicle shall be evaluated for single-lane distribution assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. For this design rating, a future wearing surface shall be considered. Load distribution for this check is based on the interior strip or interior girder and the distribution factors given in Section 17.2.7, 17.2.8, or 18.4.5.1 where applicable. See also the WisDOT policy item in 45.3.7.8.1.

For LRFR, the Wis-SPV design check shall be a permit load rating and shall be evaluated for the limit states noted in Table 45.3-1 and Table 45.3-3.

The design engineer shall check to ensure the design has a RF > 1.0 (gross vehicle load of 190 kips) for the Wis-SPV. If the design is unable to meet this minimum capacity, the engineer is required to adjust the design until the bridge can safely handle a minimum gross vehicle load of 190 kips.

Results of the Wis-SPV analysis shall be reported per 45.9.

- Bridge Rehabilitation Projects

For rehabilitation design, analysis of the Wis-SPV shall be performed as described above for new bridge construction. All efforts should be made to obtain a RF > 1.0 (gross vehicle load of 190 kips) within the confines of the scope of the project. However, it is recognized that it may not be possible to increase the Wis-SPV rating without a significant change in scope of the project. In these cases, consult the Bureau of Structures Rating Unit for further direction.

Results of the Wis-SPV analysis shall be reported per 45.9.

- Existing (In-Service) Bridges

When performing a rating for an existing (in-service) bridge, analysis of the Wis-SPV shall be performed as described above for new bridge construction. In this case – where the bridge in question is being load rated but not altered in any way – the results of the Wis-SPV analysis need simply be reported as calculated per 45.9. If the results of this analysis produce a rating factor less than 1.0 (gross vehicle load less than 190 kips), notify the Bureau of Structures Rating Unit.

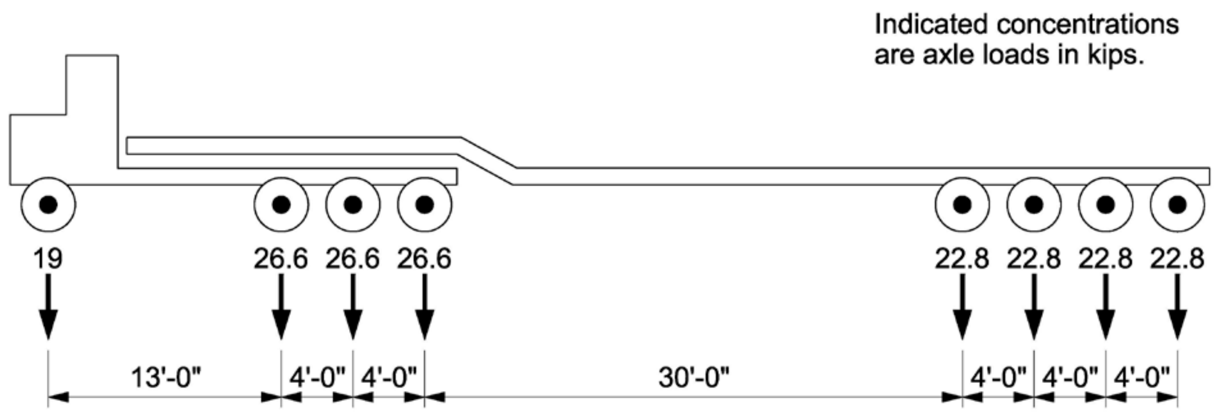


Figure 45.12-1
Wisconsin Standard Permit Vehicle (Wis-SPV)

45.13 References

1. *Final Report on Full-Scale Bridge Testing* by E. G. Burdette and D. W. Goodpasture, Department of Civil Engineer, University of Tennessee, 1971.

2. *The AASHTO Road Test*, Report 4 Bridge Research by the Highway Research Board, Washington, D.C. 1962.

3. *Standard Specifications for Highway Bridges* by American Association of State Highway and Transportation Officials.

4. *AASHTO LRFD Bridge Design Specifications* by American Association of State Highway and Transportation Officials

5. *The Manual for Bridge Evaluation, 2015 Interim Revisions* by American Association of State Highway and Transportation Officials, 2015.

6. *Structure Inspection Manual* by Wisconsin Department of Transportation, 2003.

7. *Reinforced Concrete Design* by C. K. Wang and C. G. Salmon.

8. *Plastic Design of Steel Frames* by Lynn S. Beedle.

9. National Cooperative Highway Research Program Report 312.

10. National Cooperative Highway Research Program Project 12-63.

11. *Post-Tensioning Manual* by Post-Tensioning Institute.

12. Wisconsin Statutes, Vol. 4, Chapter 348.

13. *Summary of Motor Vehicle Size and Weight Regulations in Wisconsin* by Dept. of Transportation, Division of Motor Vehicles.

14. *Evolution of Vehicular Live Load Models During the Interstate Design Era and Beyond*, John M. Kulicki and Dennis R. Mertz, Transportation Research Circular; 50 Years of Interstate Structures.

15. *Bridge Inspector's Reference Manual;* Federal Highway Administration.

16. *The Collapse of the Silver Bridge*, Chris LeRose; www.wvculture.org/history/wvhs1504.html

17. Engineering News, September 1914; L.R. Manville and R.W. Gastmeyer

18. *AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges*, by American Association of State Highway and Transportation Officials, 2003.

19. *G13.1 Guidelines for Steel Girder Bridge Analysis* by American Association of State Highway and Transportation Officials and by National Steel Bridge Alliance (NSBA), 2nd Ed., 2014.

45.14 Rating Examples

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Table of Contents

E45-1 Reinforced Concrete Slab Rating Example - LRFR

The 3-span continuous haunched slab structure shown in the Design Example from Chapter 18 is rated below. This same basic procedure is applicable for flat slab structures. For LRFR, the Bureau of Structures rates concrete slab structures for the Design Load (HL-93) and for Permit Vehicle Loads on an Interior Strip. The Permit Vehicle may be the Wisconsin Standard Permit Vehicle (Wis-SPV) or an actual Single-Trip Permit Vehicle. This bridge was analyzed using a slab width equal to one foot.

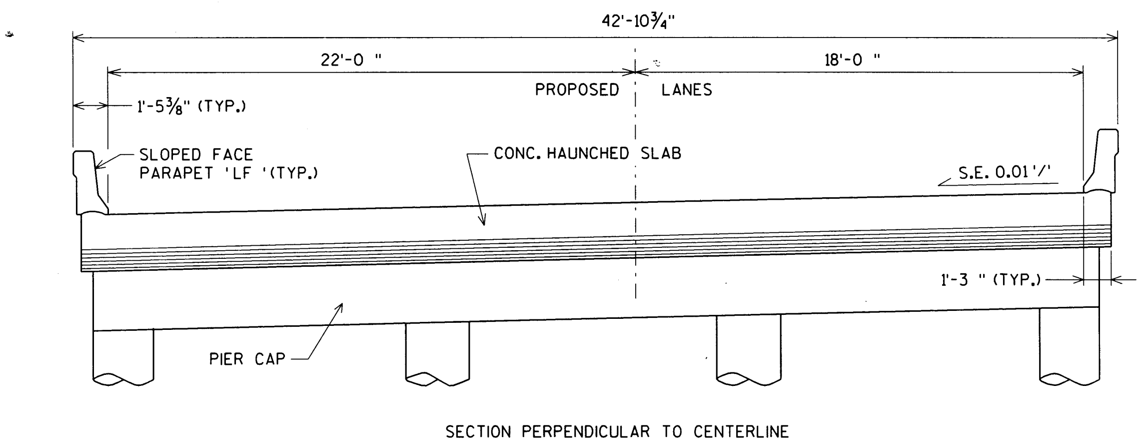

Figure E45-1.1

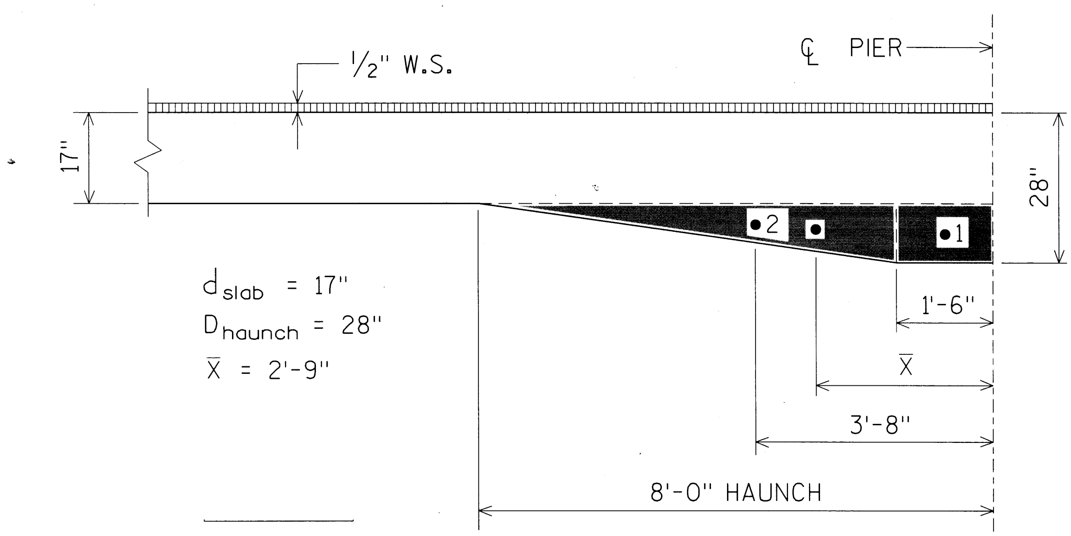

Figure E45-1.2

E45-1.1 Design Criteria

Geometry:

$L_1 := 38.0$	ft	Span 1 Length
$L_2 := 51.0$	ft	Span 2 Length
$L_3 := 38.0$	ft	Span 3 Length
$slab_{width} := 42.5$	ft	out to out width of slab
$skew := 6$	deg	skew angle (RHF)
$w_{roadway} := 40.0$	ft	clear roadway width
$cover_{top} := 2.5$	in	concrete cover on top bars (includes 1/2in wearing surface)
$cover_{bot} := 1.5$	in	concrete cover on bottom bars
$d_{slab} := 17$	in	slab depth (not including 1/2in wearing surface)
$D_{haunch} := 28$	in	haunch depth (not including 1/2in wearing surface)
$A_{st\_0.4L} := 1.71$	$\frac{in^2}{ft}$	Area of longitudinal bottom steel at 0.4L (# 9's at 7in centers)
$A_{st\_pier} := 1.88$	$\frac{in^2}{ft}$	Area of longitudinal top steel at Pier (# 8's at 5in centers)

Material Properties:

$f'_c := 4$	ksi	concrete compressive strength
$f_y := 60$	ksi	yield strength of reinforcement
$E_c := 3800$	ksi	modulus of elasticity of concrete
$E_s := 29000$	ksi	modulus of elasticity of reinforcement
$n := 8$		E_s / E_c (modular ratio)

Weights:

$w_c := 150$	pcf	concrete unit weight
$w_{LF} := 387$	plf	weight of Type LF parapet (each)

E45-1.2 Analysis of an Interior Strip - one foot width

Use Strength Limit States to rate the concrete slab bridge. **MBE [6A.4.2.2]**

The influence of ADTT and skew on force effects are ignored for slab bridges (See 18.3.2.2).

E45-1.2.1 Dead Loads (DC, DW)

The slab dead load, DC_{slab}, and the section properties of the slab, do not include the 1/2 inch wearing surface. But the 1/2 inch wearing surface load, DC_{WS}, of 6 psf must be included in the analysis of the slab. For a one foot slab width:

$DC_{WS} := 6$ 1/2 inch wearing surface load, plf

The parapet dead load is uniformly distributed over the full width of the slab when analyzing an Interior Strip. For a one foot slab width:

$$DC_{para} := 2 \cdot \frac{w_{LF}}{slab_{width}} \qquad \boxed{DC_{para} = 18} \quad \text{plf}$$

The unfactored dead load moments, M_{DC}, due to slab dead load (DC_{slab}), parapet dead load (DC_{para}), and the 1/2 inch wearing surface (DC_{ws}) are shown in Chapter 18 Example (Table E18.4).

The structure was designed for a possible future wearing surface, DW_{FWS}, of 20 psf.

$DW_{FWS} := 20$ Possible wearing surface, plf

E45-1.2.2 Live Load Distribution (Interior Strip)

Live loads are distributed over an equivalent width, E, as calculated below.
The live loads to be placed on these widths are axle loads (i.e., two lines of wheels) and the full lane load. The equivalent distribution width applies for both live load moment and shear.

Single - Lane Loading: $E = 10.0 + 5.0 \cdot (L_1 \cdot W_1)^{0.5}$ in

Multi - Lane Loading: $E = 84.0 + 1.44 \cdot (L_1 \cdot W_1)^{0.5} \le 12.0 \cdot \frac{W}{N_L}$ in

Where:

L_1 = modified span length taken equal to the lesser of the actual span or 60ft (L_1 in ft)

W_1 = modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60ft for multi-lane loading, or 30ft for single-lane loading (W_1 in ft)

W = physical edge to edge width of bridge (W in ft)

N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

For single-lane loading:

(Span 1, 3) $E := 10.0 + 5.0\cdot(38\cdot30)^{0.5}$ $\boxed{E = 178.819}$ in

(Span 2) $E := 10.0 + 5.0\cdot(51\cdot30)^{0.5}$ $\boxed{E = 205.576}$ in

For multi-lane loading:

$12.0\cdot\frac{W}{N_L} = 12.0\cdot\frac{42.5}{3} = 170$ in

(Span 1, 3) $E := 84.0 + 1.44\cdot(38\cdot42.5)^{0.5}$ $\boxed{E = 141.869}$ in <170" O.K.

(Span 2) $E := 84.0 + 1.44\cdot(51\cdot42.5)^{0.5}$ $\boxed{E = 151.041}$ in <170" O.K.

E45-1.2.3 Nominal Flexural Resistance: (M_n)

The depth of the compressive stress block, (a) is (See 18.3.3.2.1):

$$a = \frac{A_s\cdot f_s}{\alpha_1\cdot f'_c\cdot b}$$

where:

A_s = area of developed reinforcement at section (in^2)

f_s = stress in reinforcement (ksi)

$f'_c = 4$ ksi

$b := 12$ in

$\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

As shown throughout the Chapter 18 Example, when f_s is assumed to be equal to f_y, and is used to calculate (a), the value of c/d_s will be < 0.6 (for f_y = 60 ksi) per **LRFD [5.7.2.1]** Therefore the assumption that the reinforcement will yield ($f_s = f_y$) is correct. The value for (c) and (d_s) are calculated as:

$$c = \frac{a}{\beta_1}$$

$\beta_1 := 0.85$

d_s = slab depth(excl. 1/2" wearing surface) - bar clearance - 1/2 bar diameter

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals:

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

Minimum Reinforcement Check

All sections throughout the bridge meet minimum reinforcement requirements, because this was checked in the chapter 18 Design example. Therefore, no adjustment to nominal resistance (M_n) or moment capacity is required. **MBE [6A.5.6]**

E45-1.2.4 General Load - Rating Equation (for flexure)

$$RF = \frac{C - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L \cdot (M_{LL\_IM})}$$

MBE [6A.4.2.1]

For the Strength Limit State:

$$C = (\phi_c)(\phi_s)(\phi)\cdot R_n$$

where:

$R_n = M_n$ (for flexure)

$(\phi_c)(\phi_s) \geq 0.85$

Factors affecting Capacity (C):

Resistance Factor (ϕ), for Strength Limit State **MBE [6.5.3]**

$\phi := 0.9$ for flexure (all reinforced concrete section in the Chapter 18 Example were found to be tension-controlled sections as defined in **LRFD [5.7.2.1]**).

Condition Factor (ϕ_c) per Chapter 45.3.2.4

$\phi_c := 1.0$

System Factor (ϕ_s) Per Chapter 45.3.2.5

$\phi_s := 1.0$ for a slab bridge

E45-1.2.5 Design Load (HL-93) Rating

Use Strength I Limit State to find the Inventory and Operating Ratings **MBE [6A.4.2.2, 6A.5.4.1]**

Equivalent Strip Width (E) and Distribution Factor (DF):

Use the smaller equivalent width (single or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State. Multi-lane loading values will control for this bridge.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E} \qquad \text{(where E is in feet)}$$

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore is not used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

Spans 1 & 3:

DF = 1/(141"/12) = 0.0851 lanes / ft-slab

Span 2:

DF = 1/(151"/12) = 0.0795 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.
Therefore use: $DF := 0.0851$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM := 33$ % **MBE [6A.4.4.3]**

Live Loads (LL)

The live load combinations used for Strength I Limit State are shown in the Chapter 18 Example in Table E18.2 and E18.3. The unfactored moments due to Design Lane, Design Tandem, Design Truck and 90%{Double Design Truck + Design Lanes] are shown in Chapter 18 Example (Table E18.4).

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_{Li} := 1.75$ (Inventory Rating) Chapter 45 Table 45.3-1

$\gamma_{Lo} := 1.35$ (Operating Rating) Chapter 45 Table 45.3-1

The Design Load Rating was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location, for this example, is in span 1 at the 0.4 pt.

Span 1 (0.4 pt.)

Inventory:

$$RF_i = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$A_{st\_0.4L} = 1.71 \ \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 17.0 - cover_{bot} - 0.6$ $\boxed{d_s = 14.9}$ in

$a := \frac{A_{st\_0.4L}\cdot f_y}{\alpha_1 \cdot f'_c \cdot b}$ $\boxed{a = 2.51}$ in

$M_n := A_{st\_0.4L}\cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 1399.7}$ kip – in

$\boxed{M_n = 116.6}$ kip – ft

$M_{DC} := 18.1$ kip – ft (from Chapter 18 Example, Table E18.4)

$M_{DW} := 0.0$ kip – ft (additional wearing surface not for HL-93 rating runs)

The positive live load moment shall be the largest caused by the following (from Chapter 18 Example, Table E18.4):

Design Tandem (+IM) + Design Lane: (37.5 kip-ft + 7.9 kip-ft) = 45.4 kip-ft
Design Truck (+IM) + Design Lane: (35.4 kip-ft + 7.9 kip-ft) = 43.3 kip-ft

Therefore:

$M_{LL\_IM} := 45.4$ kip – ft

Inventory:

$$RF_i := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$\boxed{RF_i = 1.04}$

Operating:

$$RF_o := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Lo}\cdot(M_{LL\_IM})}$$

$\boxed{RF_o = 1.34}$

Rating for Shear:

Slab bridge designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.14.4.1]**. This bridge was designed using this procedure, therefore a shear rating is not required.

The Rating Factors, RF, for Inventory and Operating Rating are shown on the plans and also on the load rating summary sheet.

E45-1.2.6 Permit Vehicle Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.6).

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic load allowance is utilized. Future wearing surface will not be considered.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of a single-lane analysis utilizing the future wearing surface are greater than 190 kips MVW.

Use Strength II Limit State to find the Permit Vehicle Load Rating **MBE[6A.4.2.2, 6A.5.4.2.1]**.

E45-1.2.6.1 Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **MBE [6A.4.5.4.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **MBE [6A.3.2, C6A.4.5.4.2b]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E \cdot (1.20)}$$ (where E is in feet)

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33$ % **MBE [6A.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88 \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_C \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 28.0 - (cover_{top} - 0.5) - 0.5$ $\boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ in

$M_n := A_{st\_pier}\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 2720.5}$ kip – in

$\boxed{M_n = 226.7}$ kip – ft

$M_{DC} := 59.2\,kip - ft$ (from Chapter 18 Example, Table E18.4)

$M_{DW} := 1.5\ kip - ft$

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$M_{LL\_IM} := 65.2 \quad kip-ft$

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi) \cdot M_n - (\gamma_{DC}) \cdot (M_{DC}) - (\gamma_{DW}) \cdot (M_{DW})}{\gamma_L \cdot (M_{LL\_IM})}$$

$\boxed{RF_{permit} = 1.63}$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit} \cdot (190) = 310 \quad kips$ which is > 190k, Check OK

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

Rating for Shear:

WisDOT does not rate Permit Vehicles on slab bridges based on shear.

E45-1.2.6.2 Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **MBE [6A.4.5.4.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **MBE [6A.3.2, C6A.4.5.4.2b]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E \cdot (1.20)}$$ (where E is in feet)

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33$ % **MBE [6A.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88\ \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 28.0 - (cover_{top} - 0.5) - 0.5$ $\boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ in

$M_n := A_{st\_pier}\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 2720.5}$ kip – in

$\boxed{M_n = 226.7}$ kip – ft

$M_{DC} := 59.2\ kip - ft$ (from Chapter 18 Example, Table E18.4)

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$M_{LL\_IM} := 65.2$ kip – ft

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$\boxed{RF_{permit} = 1.66}$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit}\cdot(190) = 316$ kips

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

E45-1.2.6.3 Wis-SPV Permit Rating with Multi Lane Distribution w/o FWS

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

The capacity of the bridge to carry the Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is at the C/L of Pier.

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_L := 1.30$ WisDOT Policy when analyzing the Wis-SPV as an "Annual Permit" vehicle with no escorts

At C/L of Pier

Permit Vehicle:

$$RF_{permit} = \frac{(\phi_c)(\phi_s)(\phi) \cdot M_n - (\gamma_{DC}) \cdot (M_{DC})}{\gamma_L \cdot (M_{LL\_IM})}$$

$M_n = 226.7$ kip – ft (as shown previously)

$M_{DC} = 59.2$ kip – ft (as shown previously)

The live load moment at the C/L of Pier due to the Wisconsin Permit Vehicle (Wis_SPV) having a gross vehicle load of 190 kips and a DF of 0.0851 lanes/ft-slab:

$M_{LL\_IM} := 98.7$ kip – ft

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi) \cdot M_n - (\gamma_{DC}) \cdot (M_{DC})}{\gamma_L \cdot (M_{LL\_IM})}$$

$$\boxed{RF_{permit} = 1.01}$$

The Wisconsin Standard Permit Vehicle (Wis_SPV) load that can be carried by the bridge is:

$RF_{permit} \cdot (190) = 193$ kips

E45-1.3 Summary of Rating

Slab - Interior Strip							
Limit State		Design Load Rating		Legal Load Rating	Permit Load Rating (kips)		
		Inventory	Operating		Single DF w/ FWS	Single DF w/o FWS	Multi DF w/o FWS
Strength I	Flexure	1.04	1.34	N/A	310	316	193
Service I		N/A	N/A	N/A	Optional	Optional	Optional

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E45-2 Single Span PSG Bridge, LRFD Design, Rating Example - LRFR

The bridge was built in 2007 and has no deterioration. There is no overlay on the structure.

This example will perform the LRFR rating calculations for the bridge that was designed in Chapter 19 of this manual (E19-1). Though it is necessary to rate both interior and exterior girders to determine the minimum capacity, the below rating will analyze the interior girder only.

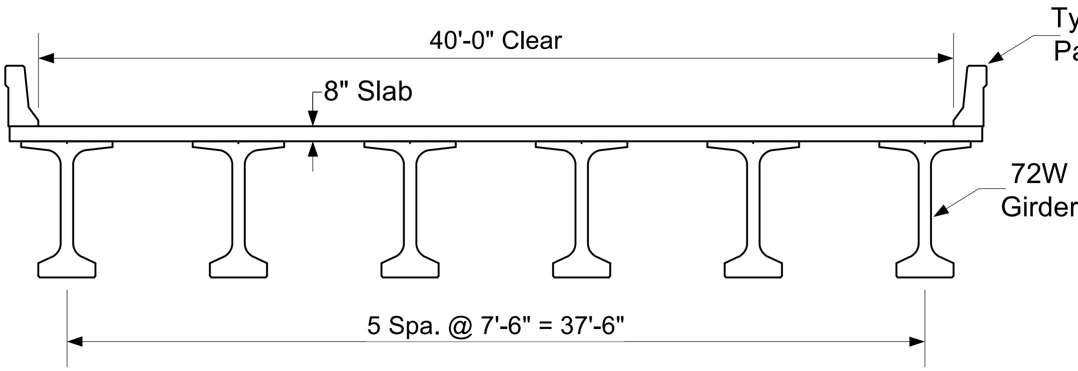


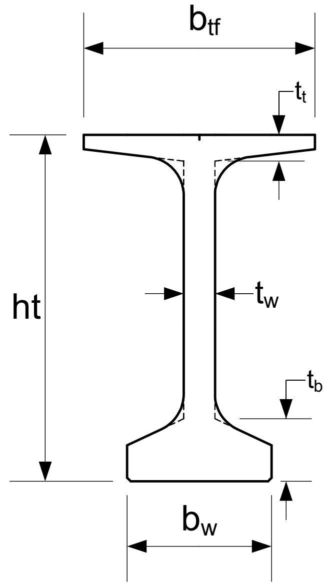
Figure E45-2.1

E45-2.1 Preliminary Data

$L := 146$	center to center of bearing, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_{pu} := 270$	strength of low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_s := 0.217$	area of strand, in2
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness (slab thickness - 1/2 in wearing surface), in
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$w_c := 0.150$	weight of concrete, kcf
$H_{avg} := 2$	average thickness of haunch, in
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$S := 7.5$	spacing of the girders, ft
$ng := 6$	number of girders

E45-2.2 Girder Section Properties

72W Girder Properties (46 strands, 8 draped):



$b_{tf} := 48$	width of top flange, in
$t_t := 5.5$	avg. thickness of top flange, in
$t_w := 6.5$	thickness of web, in
$t_b := 13$	avg. thickness of bottom flange, in
$ht := 72$	height of girder, in
$b_w := 30$	width of bottom flange, in
$A_g := 915$	area of girder, in^2
$I_g := 656426$	moment of inertia of girder, in^4
$y_t := 37.13$	centriod to top fiber, in
$y_b := -34.87$	centroid to bottom fiber, in
$S_t := 17680$	section modulus for top, in^3
$S_b := -18825$	section modulus for bottom, in^3
$w_g := 0.953$	weight of girder, klf
$ns := 46$	number of strands
$e_s := -30.52$	centriod to cg strand pattern

$$e_g := y_t + 2 + \frac{t_{se}}{2} \qquad \boxed{e_g = 42.88} \text{ in}$$

Web Depth: $d_w := ht - t_t - t_b$ $\boxed{d_w = 53.50}$ in

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351} \qquad E_B := E_{beam8}$$

$$E_{beam6.8} := 5500 \cdot \frac{\sqrt{f'_{ci} \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam6.8} = 5855} \qquad E_{ct} := E_{beam6.8}$$

$$E_D := E_{deck4}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ **LRFD [Eq 4.6.2.2.1-1]** $\boxed{K_g = 3600866}$ in^4

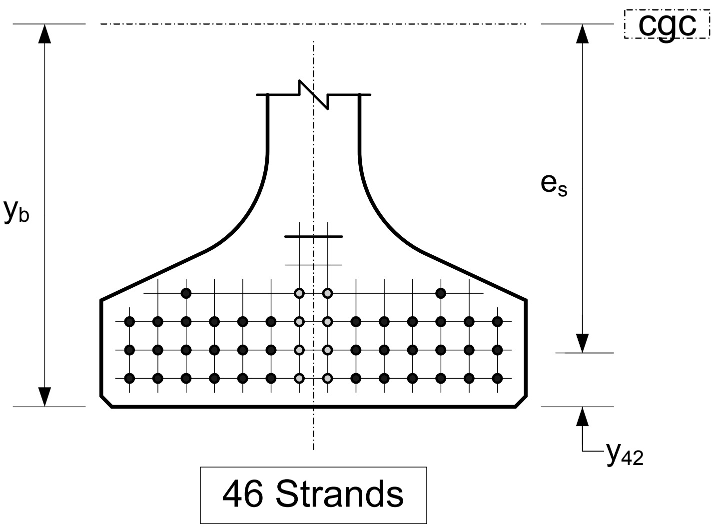


Figure E45-2.2

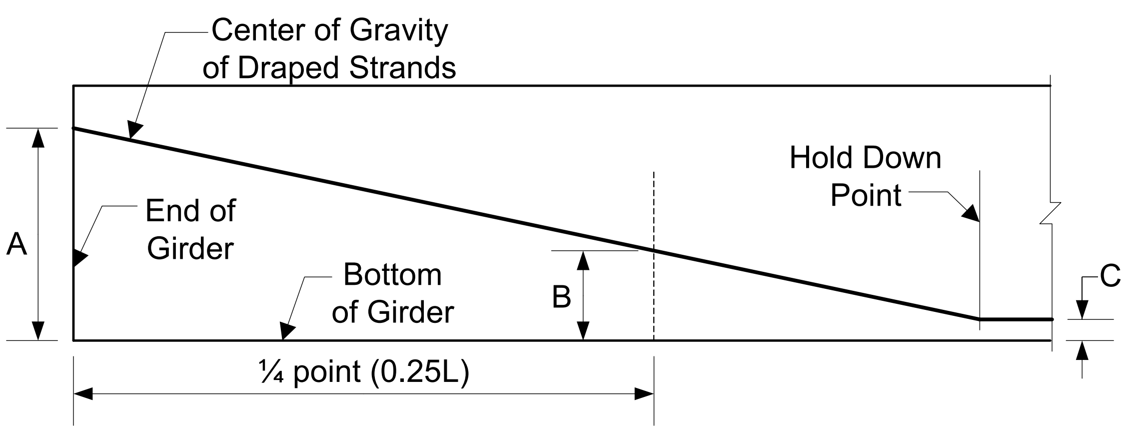


Figure E45-2.3

$A := 67$ in

$C := 5$ in

$B_{min} := 20.5$ in

$B_{max} := 23.5$ in

$$B_{avg} := \frac{B_{min} + B_{max}}{2}$$ $\boxed{B_{avg} = 22.0}$ in

$$slope := \left[\frac{A - B_{avg}}{(0.25) \cdot L \cdot 12}\right] \cdot 100$$ $\boxed{slope = 10.274}$ %

E45-2.3 Composite Girder Section Properties

Calculate the effective flange width in accordance with 17.2.11 and **LRFD [4.6.2.6]**:

$b_{eff} := S \cdot 12$ $\boxed{b_{eff} = 90.00}$ in

The effective width, b_{eff}, must be adjusted by the modular ratio, n, to convert to the same concrete material (modulus) as the girder.

$b_{eadj} := \frac{b_{eff}}{n}$ $\boxed{b_{eadj} = 58.46}$ in

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

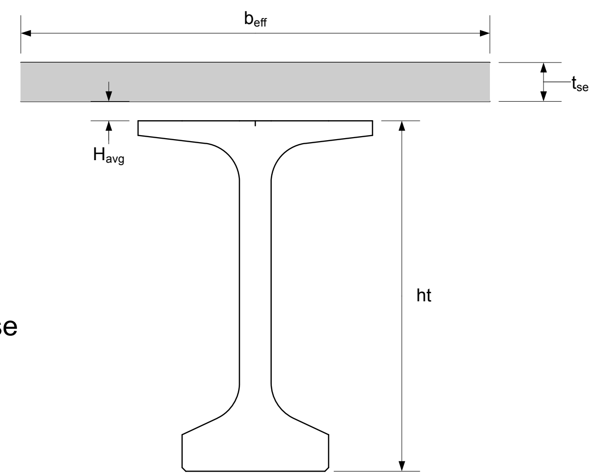
effective slab width; $\boxed{b_{eadj} = 58.46}$ in

haunch thickness; $\boxed{H_{avg} = 2.00}$ in

total height; $h_c := ht + H_{avg} + t_{se}$

$\boxed{h_c = 81.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	77.75	438	34089	2650458	2055	2652513
Girder	34.87	915	31906	1112564	656426	1768990
Haunch	73	0	0	0	0	0
Summation		1353	65996			4421503

$\Sigma A := 1353$ in^2

$\Sigma AY := 65996$ in^3

$\Sigma IplusAYsq := 4421503$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$$ $\boxed{y_{cgb} = -48.8}$ in

$$y_{cgt} := ht + y_{cgb}$$ $\boxed{y_{cgt} = 23.2}$ in

$$A_{cg} := \Sigma A$$ $\boxed{A_{cg} = 1353}$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{2}$$ $\boxed{I_{cg} = 1202381}$ in^4

$$S_{cgt} := \frac{I_{cg}}{y_{cgt}}$$ $\boxed{S_{cgt} = 51777}$ in^3

$$S_{cgb} := \frac{I_{cg}}{y_{cgb}}$$ $\boxed{S_{cgb} = -24650}$ in^3

E45-2.4 Dead Load Analysis - Interior Girder

Dead load on non-composite (DC_1):

weight of 72W girders $\boxed{w_g = 0.953}$ klf

weight of 2-in haunch

$$w_h := \left(\frac{H_{avg}}{12}\right) \cdot \left(\frac{b_{tf}}{12}\right) \cdot \left(w_c\right)$$ $\boxed{w_h = 0.100}$ klf

weight of diaphragms $\boxed{w_D := 0.006}$ klf

weight of slab

$$w_d := \left(\frac{t_s}{12}\right) \cdot (S) \cdot \left(w_c\right)$$ $\boxed{w_d = 0.750}$ ksf

$$DC_1 := w_g + w_h + w_D + w_d$$ $\boxed{DC_1 = 1.809}$ klf

$$V_{DC1} := \frac{DC_1 \cdot L}{2}$$ $\boxed{V_{DC1} = 132}$ kips

$$M_{DC1} := \frac{DC_1 \cdot L^2}{8}$$ $\boxed{M_{DC1} = 4820}$ kip-ft

* Dead load on composite (DC_2):

weight of single parapet, klf $\boxed{w_p = 0.387}$ klf

weight of 2 parapets, divided equally to all girders, klf

$$DC_2 := \frac{w_p \cdot 2}{ng} \qquad \boxed{DC_2 = 0.129} \text{ klf}$$

$$V_{DC2} := \frac{DC_2 \cdot L}{2} \qquad \boxed{V_{DC2} = 9} \text{ kips}$$

$$M_{DC2} := \frac{DC_2 \cdot L^2}{8} \qquad \boxed{M_{DC2} = 344} \text{ kip-ft}$$

* Wearing Surface (DW): There is no current wearing surface on this bridge. However, it is designed for a 20 psf future wearing surface. Thus, it will be used in the calculations for the Wisconsin Standard Permit Vehicle Design Check, Section 45.12.

$$DW := \frac{w \cdot 0.020}{ng} \qquad \boxed{DW = 0.133} \text{ klf}$$

$$V_{DW} := \frac{DW \cdot L}{2} \qquad \boxed{V_{DW} = 10} \text{ kips}$$

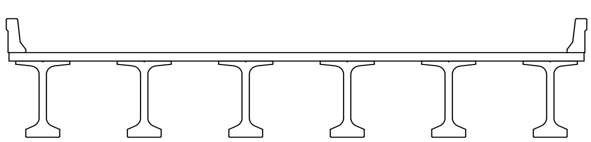
$$M_{DW} := \frac{DW \cdot L^2}{8} \qquad \boxed{M_{DW} = 355} \text{ kip-ft}$$

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E45-2.5 Live Load Analysis - Interior Girder

Live Load Distribution Factors (g)

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

E45-2.5.1 Moment Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i1} = 0.435}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i2} = 0.636}$$

$$g_i := \max\left(g_{i1}, g_{i2}\right) \qquad \boxed{g_i = 0.636}$$

Note: The distribution factors above already have a multiple presence factor included that is used for service and strength limit states. For permit load analysis utilizing single lane distribution, the 1.2 multiple presence factor should be divided out.

E45-2.5.2 Shear Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{v1} := 0.36 + \frac{S}{25} \qquad \boxed{g_{v1} = 0.660}$$

Two or More Lanes Loaded:

$$g_{v2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \boxed{g_{v2} = 0.779}$$

$$g_v := \max\left(g_{v1}, g_{v2}\right) \qquad \boxed{g_v = 0.779}$$

E45-2.5.3 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the dynamic load allowance is applied only to the truck portion of the HL-93 loads.

Unfactored Live Load + Impact Moments per Lane (kip-ft)		
Tenth Point	Truck	Tandem
0	0	0
0.1	1783	1474
0.2	2710	2618
0.3	4100	3431
0.4	4665	3914
0.5	4828	4066

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$g_i = 0.636$

$M_{LLIM} := g_i \cdot 4828$ $\boxed{M_{LLIM} = 3073}$ kip-ft

E45-2.6 Compute Nominal Flexural Resistance at Midspan

At failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD Table [C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assumed dimensions:

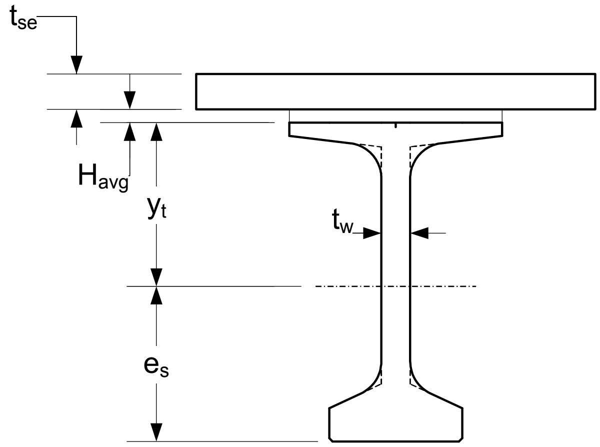


Figure E45-2.4

Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := ns \cdot A_s$ $\boxed{A_{ps} = 9.98}$ in^2

$b := b_{eff}$ $\boxed{b = 90.00}$ in

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \leq 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_{cd} - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.850}$

$d_p := y_t + H_{avg} + t_{se} - e_s$ $\boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.99}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.49}$ in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$ depth of compression flange $\boxed{t_{se} = 7.500}$ in

$b_{tf} = 48.00$ width of top flange, inches

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c = 10.937} \text{ in}$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 9.30} \text{ in}$$

This is above the base of the haunch (9.5 inches) and nearly to the web of the girder. Assume OK.
Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) \qquad \boxed{f_{ps} = 259.283} \text{ ksi}$$

$$T_u := f_{ps} \cdot A_{ps} \qquad \boxed{T_u = 2588} \text{ kips}$$

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.7.3.2], [5.7.3.2.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_n = 15717} \text{ kip-ft}$$

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$$M_r := \phi_f \cdot M_n \qquad \boxed{M_r = 15717} \text{ kip-ft}$$

Check Minimum Reinforcement
The amount of reinforcement must be sufficient to develop M_r equal to the lesser of M_{cr} or 1.33 M_u per **LRFD [5.7.3.3.2]**

$$\gamma_{LL} := 1.75 \qquad \gamma_{DC} = 1.250 \qquad \eta := 1.0$$

$$Mu := \eta \cdot \left[\gamma_{DC} \cdot (M_{DC1} + M_{DC2}) + \gamma_{LL} \cdot M_{LLIM}\right] \qquad \boxed{Mu = 11832} \text{ kip-ft}$$

$$\boxed{1.33 \cdot Mu = 15737} \text{ kip-ft}$$

Calculate M_{cr} next and compare its value with 1.33 Mu

M_{cr} is calculated as follows:

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.679}$ ksi

$f_{cpe} := \dfrac{T}{A_g} + \dfrac{T \cdot e_s}{S_b}$ $\boxed{f_{cpe} = 4.414}$ ksi

$M_{dnc} := M_{DC1}$ $\boxed{M_{dnc} = 4820}$ kip-ft

$S_c := -S_{cgb}$ $\boxed{S_c = 24650}$ ksi

$S_{nc} := -S_b$ $\boxed{S_{nc} = 18825}$ ksi

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete members

$M_{cr} := \gamma_3 \cdot \left[S_c \cdot (\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}) \cdot \dfrac{1}{12} - M_{dnc} \cdot \left(\dfrac{S_c}{S_{nc}} - 1 \right) \right]$ $\boxed{M_{cr} = 10713}$ kip-ft

$M_{cr} = 10713$ kip-ft < $1.33Mu = 15737$, therefore M_{cr} controls

This satisfies the minimum reinforcement check since $M_{cr} < M_r$

Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$T_{oi} := ns \cdot f_{tr} \cdot A_s$ $\boxed{= 46 \cdot 202.5 \cdot 0.217 = 2021}$ kips

The ES loss estimated above was: $\Delta f_{pES\_est} := 17$ ksi, or $ES_{loss} = 7.900$ %. The resulting force in the strands after ES loss:

$T_o := \left(1 - \dfrac{ES_{loss}}{100} \right) \cdot T_{oi}$ $\boxed{T_o = 1862}$ kips

If we assume all strands are straight we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A_g} + \left(T_o \cdot e_s\right) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g}$$ $\boxed{f_{cgp} = 3.240}$ ksi

$\boxed{E_{ct} = 5855}$ ksi

$$E_p := E_s$$ $\boxed{E_p = 28500}$ ksi

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp}$$ $\boxed{\Delta f_{pES} = 15.771}$ ksi

$$f_i := f_{tr} - \Delta f_{pES}$$ $\boxed{f_i = 186.729}$ ksi

Approximate Estimate of Time Dependant Losses

Calculate the components of the time dependant losses; shrinkage, creep and relaxation, using the approximate method in accordance with **LRFD [5.9.5.3]**.

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pi} \cdot A_s}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

From **LRFD [Figure 5.4.2.3.3-1]**, the average annual ambient relative humidity, $H := 72$ %.

$$\gamma_h := 1.7 - 0.01 \cdot H$$ $\boxed{\gamma_h = 0.980}$

$$\gamma_{st} := \frac{5}{1 + f'_{ci}}$$ $\boxed{\gamma_{st} = 0.641}$

$\Delta f_{pR} := 2.4$ ksi for low relaxation strands

$$\Delta f_{pCR} := 10.0 \cdot \frac{f_{tr} \cdot A_s \cdot ns}{A_g} \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pCR} = 13.878}$ ksi

$$\Delta f_{pSR} := 12.0 \cdot \gamma_h \cdot \gamma_{st}$$ $\boxed{\Delta f_{pSR} = 7.538}$ ksi

$$\Delta f_{pRE} := \Delta f_{pR}$$ $\boxed{\Delta f_{pRE} = 2.400}$ ksi

$$\Delta f_{pLT} := \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pRE}$$ $\boxed{\Delta f_{pLT} = 23.816}$ ksi

The total estimated prestress loss (Approximate Method):

$\Delta f_p := \Delta f_{pES} + \Delta f_{pLT}$ $\boxed{\Delta f_p = 39.587}$ ksi

$\boxed{\frac{\Delta f_p}{f_{tr}} \cdot 100 = 19.549}$ % total prestress loss

The remaining stress in the strands and total force in the beam after all losses is:

$f_{pe} := f_{tr} - \Delta f_p$ $\boxed{f_{pe} = 162.91}$ ksi

E45-2.7 Compute Nominal Shear Resistance at First Critical Section

Note: **MBE [6A.5.8]** does not require a shear evaluation for the Design Load Rating or the Legal Load Rating provided the bridge shows no visible sign of shear distress. However, for this example, we will show one iteration for the Design Load Rating.

The shear analysis is always required for Permit Load Rating.

The following will illustrate the calculation at the first critical section only. Due to the variation of resistances for shear along the length of the prestressed concrete I-beam, it is not certain what location will govern. Therefore, a systematic evaluation of the shear and the longitudinal yield criteria based on shear-moment interation should be performed along the length of the beam.

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := t_w$ $\boxed{b_v = 6.50}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.
d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$d_v := -e_s + y_t + H_{avg} + t_{se} - \frac{a}{2}$ $\boxed{d_v = 72.50}$ in

However, since there are draped strands for a distance of $HD := 49$ from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section. Since the draped strands will raise the center of gravity of the strand group near the girder end, try a smaller value of "d_v" and recalculate "e_s" and "a".

Try $d_v := 65$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right)\cdot\frac{1}{12} + 0.5 \qquad \boxed{L_{crit} = 6.25} \text{ ft}$$

Calculate the eccentricity of the strand group at the critical section.

$slope = 10.274$

$y_{8t} := A + y_b$

$y_{8t} = 32.130$

$ns_{sb} := 38$ number of undraped strands

$ns_d := 8$ number of draped strands

Find the center of gravity for the 38 straight strands from the bottom of the girder:

$$Y_S := \frac{12\cdot 2 + 12\cdot 4 + 12\cdot 6 + 2\cdot 8}{ns_{sb}} \qquad \boxed{Y_S = 4.211} \text{ in}$$

$$y_s := y_b + Y_S \qquad y_s = -30.659 \text{ in}$$

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100}\cdot L_{crit}\cdot 12 \qquad \boxed{y_{8t\_crit} = 24.42} \text{ in}$$

$$e_{s\_crit} := \frac{ns_{sb}\cdot y_s + ns_d\cdot y_{8t\_crit}}{ns_{sb} + ns_d} \qquad \boxed{e_{s\_crit} = -21.08} \text{ in}$$

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + H_{avg} + t_{se} - e_{s\_crit} \qquad \boxed{d_{p\_crit} = 67.71} \text{ in}$$

Note that the area of steel is based on the number of bonded strands.

$$A_{ps\_crit} := (ns)\cdot A_s \qquad \boxed{A_{ps\_crit} = 9.98} \text{ in}^2$$

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$\boxed{d_b = 0.600}$ in

$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b$ $\boxed{l_d = 144.6}$ in

The transfer length may be taken as: $l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.250$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} + \frac{L_{crit} \cdot 12 - l_{tr}}{l_d - l_{tr}} \cdot \left(f_{ps} - f_{pe}\right) \qquad \boxed{f_{pu\_crit} = 198} \text{ ksi}$$

For rectangular section behavior:

$$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \frac{f_{pu\_crit}}{d_{p\_crit}}} \qquad \boxed{c = 7.349} \text{ in}$$

$$a_{crit} := \beta_1 \cdot c \qquad \boxed{a_{crit} = 6.247} \text{ in}$$

Calculation of shear depth based on refined calculations of e_s and a:

$$d_{v\_crit} := -e_{s\_crit} + y_t + H_{avg} + t_{se} - \frac{a_{crit}}{2} \qquad \boxed{d_{v\_crit} = 64.59} \text{ in}$$

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**).

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kips)

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kips). (Not necessarily equal to V_u.)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in)

M_{max} = maximum factored moment at section due to externally applied loads (kip-in)

M_{dnc} = total unfactored dead load moment acting on the noncomposite section (kip-ft)

Values for the following moments and shears are at the critical section, $L_{crit} = 6.25$ feet from the end of the girder at the abutment.

$\boxed{V_{DCnc} = 121.7}$ kips

$\boxed{V_{DCc} = 8.7}$ kips

$\boxed{V_{DWc} = 9.0}$ kips

$Vi_{LL} := Vi_{LL\_lane} \cdot g_{vi}$ $\boxed{Vi_{LL} = 100.5}$ kips

$V_i := 1.75 \cdot Vi_{LL}$ $\boxed{V_i = 175.9}$ kips

$V_d := V_{DCc} + V_{DCnc} + V_{DWc}$ $\boxed{V_d = 139.3}$ kips

$V_u := 1.25 \cdot (V_{DCnc} + V_{DCc}) + 1.5 \cdot V_{DWc} + 1.75 \cdot Vi_{LL}$ $\boxed{V_u = 352.2}$ kips

$M_{dnc} := 730$ kip-ft

$M_{max} := 837$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$\boxed{M_{max} = 10044}$ kip-in

$f_r = -0.20 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := -0.20 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = -0.566}$ ksi

$T_{crit} := A_{ps\_crit} \cdot f_{pe}$ $\boxed{T_{crit} = 1626}$ kips

$f_{cpe} := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_b}$ $\boxed{f_{cpe} = 3.598}$ ksi

$\boxed{M_{dnc} = 730}$ kip-ft

$\boxed{M_{max} = 10044}$ kip-in

$S_c := S_{cgb}$ $\boxed{S_c = -24650}$ in^3

$S_{nc} := S_b$ $\boxed{S_{nc} = -18825}$ in^3

$M_{cre} := S_c \cdot \left(f_r - f_{cpe} - \frac{12M_{dnc}}{S_{nc}} \right)$ $\boxed{M_{cre} = 91171}$ kip-in

Calculate V_{ci}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{ci1} := 0.06 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_{ci1} = 71.7}$ kips

$V_{ci2} := 0.02 \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}$ $\boxed{V_{ci2} = 1759.7}$ kips

$V_{ci} := \max(V_{ci1}, V_{ci2})$ $\boxed{V_{ci} = 1759.7}$ kips

$f_t := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_t} + \frac{M_{dnc} \cdot 12}{S_t}$ $\boxed{f_t = 0.334}$ ksi

$f_b := \frac{T_{crit}}{A_g} + \frac{T_{crit} \cdot e_{s\_crit}}{S_b} + \frac{M_{dnc} \cdot 12}{S_b}$ $\boxed{f_b = 3.133}$ ksi

$\boxed{y_{cgb} = -48.78}$ in

$\boxed{ht = 72.00}$ in

$f_{pc} := f_b - y_{cgb} \cdot \frac{f_t - f_b}{ht}$ $\boxed{f_{pc} = 1.237}$ ksi

$V_{p\_cw} := ns_d \cdot A_s \cdot f_{pe} \cdot \frac{slope}{100}$ $\boxed{V_{p\_cw} = 29.1}$ kips

Calculate V_{cw}, **LRFD [5.8.3.4.3]** λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_{cw} := \left(0.06 \cdot \lambda \sqrt{f'_c} + 0.30 \cdot f_{pc}\right) \cdot b_v \cdot d_v + V_{p\_cw}$ $\boxed{V_{cw} = 257.5}$ kips

$V_c := \min(V_{ci}, V_{cw})$ $\boxed{V_c = 257.5}$ kips

Calculate the shear resistance at L_{crit}:

$\phi_v := 0.9$ **LRFD [5.5.4.2]**

$s := 20$ in

$A_v := 0.40$ in^2 for #4 rebar

$f_y := 60$ ksi

$d_v = 65.00$ in

$$\cot\theta := \begin{cases} 1 & \text{if } V_{ci} < V_{cw} \\ \min\left(1.0 + 3\cdot\dfrac{f_{pc}}{\sqrt{f'_c}}, 1.8\right) & \text{otherwise} \end{cases}$$

$\cot\theta = 1.800$

$$V_s := A_v \cdot f_y \cdot d_v \cdot \frac{\cot\theta}{s}$$

LRFD Eq 5.8.3.3-4 reduced per **C5.8.3.3-1** when α = 90 degrees.

$V_s = 140$ kips

$Vn_1 := V_c + V_s + V_p$ $Vn_1 = 398$ kips

$Vn_2 := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $Vn_2 = 845$ kips

$V_n := \min(Vn_1, Vn_2)$ $V_n = 398$ kips

$V_r := \phi_v \cdot V_n$ $V_r = 358.11$ kips

E45-2.8 Longitudinal Tension Flange Capacity:

The total capacity of the tension reinforcing must meet the requirements of **LRFD [5.8.3.5].** The capacity is checked at the critical section for shear:

$$V_u := 1.25\cdot(V_{DC1} + V_{DC2}) + 1.50\cdot(V_{DW}) + 1.75\cdot(Vu_{LL})$$

$V_u = 367.320$ kips

$$T_{ps} := \frac{M_{max}}{d_v \cdot \phi_f} + \left(\frac{V_u}{\phi_v} - 0.5\cdot V_s - V_{p\_cw}\right)\cdot\cot\theta$$

$T_{ps} = 711$ kips

actual capacity of the straight bonded strands:

$\boxed{ns_{sb} \cdot A_s \cdot f_{pu\_crit} = 1629}$ kips

Is the capacity of the straight bonded strands greater than T_{ps}? $\boxed{check = \text{"OK"}}$

Check the tension Capacity at the edge of the bearing:

The strand is anchored $l_{px} := 10$ inches. The transfer and development lengths for a prestressing strand are calculated in accordance with **LRFD [5.11.4.2]**:

$\boxed{l_{tr} = 36.00}$ in

$\boxed{l_d = 144.6}$ in

Since l_{px} is less than the transfer length, the design stress in the prestressing strand is calculated as follows:

The assumed crack plane crosses the centroid of the straight strands at

$l_{px'} := l_{px} + Y_S \cdot \cot\theta$ $\boxed{Y_S = 4.211}$ in $\boxed{l_{px'} = 17.58}$ in

$f_{pb} := \dfrac{f_{pe} \cdot l_{px'}}{60 \cdot d_b}$ $\boxed{f_{pb} = 79.55}$ kips

Tendon capacity of the straight bonded strands: $\boxed{ns_{sb} \cdot A_s \cdot f_{pb} = 656}$ kips

The values of V_u, V_s, V_p and θ may be taken at the location of the critical section.

Over the length d_v, the average spacing of the stirrups is:

$s_{ave} := \dfrac{6 \cdot 4.5 + 3 \cdot s}{9}$ $\boxed{s_{ave} = 9.67}$ in

$V_s := A_v \cdot f_y \cdot d_v \cdot \dfrac{\cot\theta}{s_{ave}}$ $\boxed{V_s = 290}$ kips

The vertical component of the draped strands is: $\boxed{V_{p\_cw} = 29}$ kips

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 352}$ kips

E45-2.9 Design Load Rating

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Live Load Factors taken from Table 45.3-1

$\gamma_{L\_inv} := 1.75$ $\gamma_{DC} := 1.25$ $\gamma_{servLL} := 0.8$

$\gamma_{L\_op} := 1.35$ $\phi_c := 1.0$ $\phi_s := 1.0$

$\phi := 1.0$ for flexure

$\phi := 0.9$ for shear

For Flexure

Inventory Level

$$RF_{Mom\_Inv} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_inv} \cdot (M_{LLIM})}$$

$$\boxed{RF_{Mom\_Inv} = 1.723}$$

Operating Level

$$RF_{Mom\_Op} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_op} \cdot (M_{LLIM})}$$

$$\boxed{RF_{Mom\_Op} = 2.233}$$

For Shear at first critical section

Inventory Level

$$RF_{shear\_Inv} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC} \cdot (V_{DCnc} + V_{DCc})}{\gamma_{L\_inv} \cdot (Vi_{LL})}$$

$$\boxed{RF_{shear\_Inv} = 1.110}$$

Operating Level

$$RF_{shear\_Op} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC} \cdot (V_{DCnc} + V_{DCc})}{\gamma_{L\_op} \cdot (Vi_{LL})}$$

$$\boxed{RF_{shear\_Op} = 1.439}$$

At the Service III Limit State (Inventory Level):

$$RF = \frac{f_R - \gamma_D \cdot (f_D)}{\gamma_{servLL}(f_{LLIM})}$$

$T := ns \cdot A_s \cdot f_{pe}$ $\boxed{T = 1626}$ kips

$f_{pb} := \dfrac{T}{A_g} + \dfrac{T \cdot (e_s)}{S_b}$ $\boxed{f_{pb} = 4.414}$ ksi

Allowable Tensile Stress **LRFD [5.9.4.2.2]**

$t_{all} = -0.19 \cdot \lambda \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$t_{all} := -0.19 \cdot \sqrt{f'_c}$; | t_{all} | ≤ 0.6 ksi $\boxed{t_{all} = -0.537}$ ksi

$f_R := f_{pb} - t_{all}$ $\boxed{f_R = 4.951}$ ksi

Live Load Stresses:

$f_{LLIM} := \dfrac{M_{LLIM} \cdot 12}{S_{cgb}}$ $\boxed{f_{LLIM} = 1.496}$ ksi

Dead Load Stresses:

$f_{DL} := \dfrac{M_{DC1} \cdot 12}{S_b} + \dfrac{M_{DC2} \cdot 12}{S_{cgb}}$ $\boxed{f_{DL} = 3.240}$ ksi

$RF_{serviceIII} := \dfrac{f_R - 1.0 \cdot (f_{DL})}{\gamma_{servLL} \cdot (f_{LLIM})}$ $\boxed{RF_{serviceIII} = 1.430}$

E45-2.10 Legal Load Rating

Since the Operating Design Load Rating RF>1.0, the Legal Load Rating is not required. The Legal Load computations that follow have been done for illustrative purposes only. Shear ratings have not been illustrated.

Live Loads used will be the AASHTO Legal Loads per Figure 45.10-1 and AASHTO Specialized Hauling Vehicles per Figure 45.10-2.

$\boxed{g_i = 0.636}$

$IM := 33$ %

* WisDOT does not allow for a dynamic load allowance reduction based on the smoothness of the roadway surface. Thus, IM=33%

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Live Load Factors taken from Tables 45.3-1 and 45.3-2

$\phi_c := 1.0$ $\phi_s := 1.0$

$\phi := 1.0$

$\gamma_{L\_Legal} := 1.45$ $\gamma_{DC} := 1.25$

$\gamma_{L\_SU} := 1.45$

For Flexure

$$RF_{Legal} := \frac{(1)(1)(1)(M_n) - \gamma_{DC}\cdot(M_{DC1} + M_{DC2})}{\gamma_{L\_Legal}\cdot(M_{LLIM})}$$

$$RF_{SU} := \frac{(1)(1)(1)(M_n) - \gamma_{DC}\cdot(M_{DC1} + M_{DC2})}{\gamma_{L\_SU}\cdot(M_{LLIM})}$$

AASHTO Type	Truck Type	Truck Weight (Tons)	M_{LL} (Per Lane) (ft-kips)	M_{LLIM} (M_{LL} * IM * g_i) ft-kips	RF Strength I Flexure	Safe Load Capacity (Tons)	Posting?
Commercial Trucks	Type 3	25	1671.0	1413.4	4.520	113	No
	Type 3S2	36	2150.0	1818.6	3.513	126	No
	Type 3-3	40	2260.0	1911.7	3.342	134	No
Specialized Hauling Vehicles	SU4	27	1831.0	1548.8	4.124	111	No
	SU5	31	2062.8	1744.9	3.661	113	No
	SU6	34.75	2294.6	1940.9	3.291	114	No
	SU7	38.75	2540.8	2149.2	2.972	115	No

As expected, all rating factors are well above 1.0. However, if any of the rating factors would have fallen below 1.0, the posting capacity would have been calculated per 45.10.3.2:

$$\text{Posting} := \left(\frac{W}{0.7}\right)[(RF) - 0.3]$$

E45-2.11 Permit Load Rating

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per 45.12.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic allowance is utilized. Future wearing surface shall be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW.

Also, divide out the 1.2 multiple presence factor per **MBE [6A.4.5.4.2]** for the single lane distribution factor run.

For 146' span:

$M190_{LL} := 4930.88$ kip-ft per lane

$V190_{LL} := 145.08$ kips at $\boxed{d_v = 65}$ in

for Strength Limit State

Single Lane Distribution w/ Future Wearing surface (Design check per 45.12)

$g_{m1} := 0.435 \frac{1}{1.2}$ $\boxed{g_{m1} = 0.363}$

$g_{v1} := .660 \cdot \frac{1}{1.2}$ $\boxed{g_{v1} = 0.550}$

For flexure:

$M_{190LLIM} := M190_{LL} \cdot g_{m1} \cdot 1.33$ $\boxed{M_{190LLIM} = 2377}$ kip-ft

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25 \cdot (M_{DC1} + M_{DC2}) - 1.5 \cdot (M_{DW})}{1.2(M_{190LLIM})}$$

$\boxed{RF_{190\_moment} = 3.060}$

$Wt := RF_{190\_moment} \cdot 190$ $\boxed{Wt = 581}$ kips >> 190 kips, OK

For shear:

$V_{190LLIM} := V190_{LL} \cdot g_{v1} \cdot 1.33$ $\boxed{V_{190LLIM} = 106}$ kips

$$RF_{190\_shear} := \frac{[(1)(1)(0.9)V_n] - 1.25 \cdot (V_{DCnc} + V_{DCc}) - 1.5(V_{DW})}{1.2(V_{190LLIM})}$$

$\boxed{RF_{190\_shear} = 1.418}$

$Wt := RF_{190\_shear} \cdot 190$ $\boxed{Wt = 269}$ kips > 190 kips, OK

Single Lane Distribution w/o Future Wearing surface (For plans and rating sheet only)

$g_{m1} := 0.435 \frac{1}{1.2}$ $\boxed{g_{m1} = 0.363}$

$g_{v1} := .660 \cdot \frac{1}{1.2}$ $\boxed{g_{v1} = 0.550}$

For flexure:

$M_{190LLIM} := M190_{LL} \cdot g_{m1} \cdot 1.33$ $\boxed{M_{190LLIM} = 2377}$ kip-ft

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25 \cdot (M_{DC1} + M_{DC2})}{1.2(M_{190LLIM})}$$

$\boxed{RF_{190\_moment} = 3.247}$

$Wt := RF_{190\_moment} \cdot 190$ $\boxed{Wt = 617}$

For shear:

$V_{190LLIM} := V190_{LL} \cdot g_{v1} \cdot 1.33$ $\boxed{V_{190LLIM} = 106}$ kips

$$RF_{190\_shear} := \frac{[(1)(1)(0.9)V_n] - 1.25 \cdot (V_{DCnc} + V_{DCc})}{1.2(V_{190LLIM})}$$

$\boxed{RF_{190\_shear} = 1.533}$

$Wt := RF_{190\_shear} \cdot 190$ $\boxed{Wt = 291}$

Multi-Lane Distribution w/o Future Wearing Surface (For plans and rating sheet only)

$g_{m2} := 0.636$ $\boxed{g_{m2} = 0.636}$

$g_{v2} := .779$ $\boxed{g_{v2} = 0.779}$

For flexure:

$M_{190LLIM} := M190_{LL} \cdot g_{m2} \cdot 1.33$ $\boxed{M_{190LLIM} = 4171}$ kip-ft

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25 \cdot (M_{DC1} + M_{DC2})}{1.3(M_{190LLIM})}$$

$\boxed{RF_{190\_moment} = 1.708}$

$Wt := RF_{190\_moment} \cdot 190$ $\boxed{Wt = 325}$

For shear:

$V_{190LLIM} := V190_{LL} \cdot g_{v2} \cdot 1.33$ $\boxed{V_{190LLIM} = 150}$ kips

$$RF_{190\_shear} := \frac{(1)(1)(0.9)V_n - 1.25 \cdot (V_{DCnc} + V_{DCc})}{1.3(V_{190LLIM})}$$

$\boxed{RF_{190\_shear} = 0.999}$

$Wt := RF_{190\_shear} \cdot 190$ $\boxed{Wt = 190}$

E45-2.12 Summary of Rating Factors

Interior Girder							
Limit State		Design Load Rating		Legal Load Rating	Permit Load Rating (kips)		
		Inventory	Operating		Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Strength I	Flexure	1.723	2.233	N/A	581	617	325
	Shear	1.11	1.439	N/A	269	291	190
Service III		1.43	N/A	N/A	N/A	N/A	N/A
Service I		N/A	N/A	N/A	Optional	Optional	Optional

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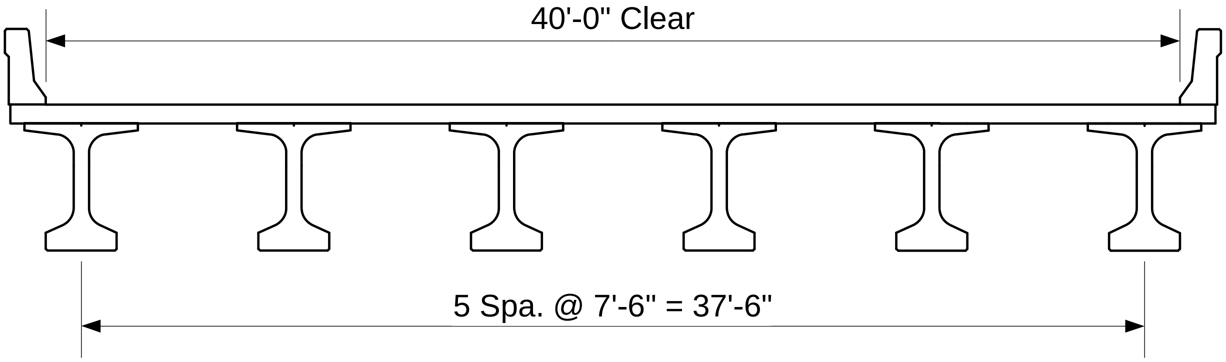
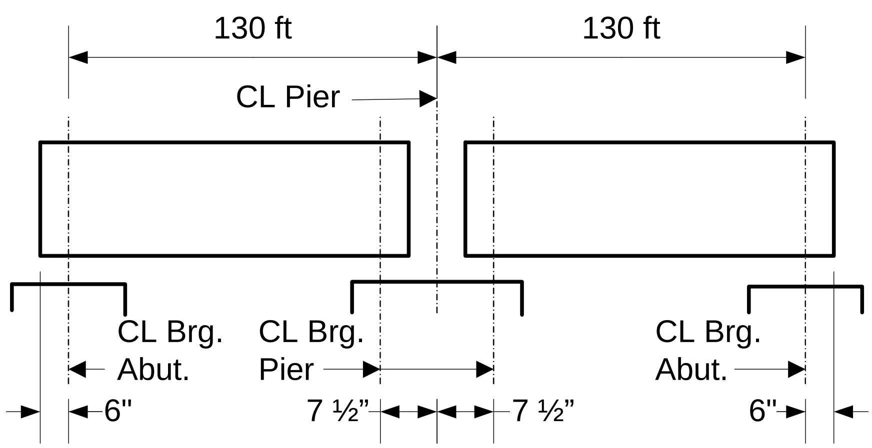
E45-3 Two Span 54W" Prestressed Girder Bridge - Continuity Reinforcement, LRFD Design, Rating Example - LRFR

This example will perform the LRFR rating calcualtions for the bridge that was designed in Chapter 19 of this manual (E19-2). Though it is necessary to rate both the interior and exterior girders to determine the minimum capacity, this example will analyze the interior girder only in the negative moment region (continuity reinforcement).





E45-3.1 Design Criteria

$L := 130$	center of bearing at abutment to CL pier for each span, ft
$L_g := 130.375$	total length of the girder (the girder extends 6 inches past the center of bearing at the abutment and 1.5" short of the center line of the pier).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_y := 60$	yield strenght of mild reinforcement, ksi

$E_s := 29000$	ksi, Modulus of Elasticity of the reinforcing steel
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$skew := 0$	skew angle, degrees
$w_c := 0.150$	kcf
$h := 2$	height of haunch, inches

E45-3.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351}$$

$$E_B := E_{beam8}$$

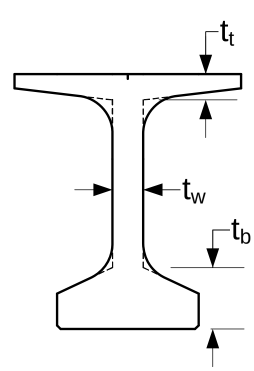
$$E_D := E_{deck4}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

E45-3.3 Section Properties

54W Girder Properties:

$w_{tf} := 48$	in
$t_w := 6.5$	in
$ht := 54$	in
$b_w := 30$	width of bottom flange, in
$A_g := 798$	in²
$I_g := 321049$	in⁴
$y_t := 27.70$	in
$y_b := -26.30$	in



E45-3.4 Girder Layout

$S := 7.5$	Girder Spacing, feet
$s_{oh} := 2.50$	Deck overhang, feet
$ng := 6$	Number of girders

E45-3.5 Loads

$w_g := 0.831$	weight of 54W girders, klf
$w_d := 0.100$	weight of 8-inch deck slab (interior), ksf
$w_h := 0.100$	weight of 2-in haunch, klf
$w_{di} := 0.410$	weight of each diaphragm on interior girder (assume 2), kips
$w_{ws} := 0.020$	future wearing surface, ksf
$w_p = 0.387$	weight of parapet, klf

E45-3.5.1 Dead Loads

Dead load on non-composite (DC):

interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L} \qquad \boxed{w_{dlii} = 1.687} \text{ klf}$$

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng} \qquad \boxed{w_p = 0.129} \text{ klf}$$

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng} \qquad \boxed{w_{ws} = 0.133} \text{ klf}$$

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E45-3.5.2 Live Loads

For Stength 1 and Service 1:

HL-93 loading = truck + lane **LRFD [3.6.1.3.1]**

truck pair + lane

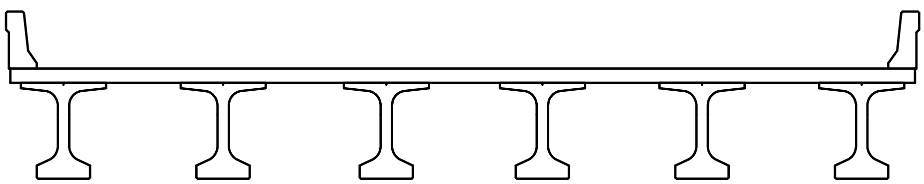
DLA of 33% applied to truck or tandem, but not to lane per **LRFD [3.6.2.1].**

For Fatigue:

HL-93 truck (no lane) with 15% DLA and 30 ft rear axle spacing per **LRFD [3.6.1.4.1].**

E45-3.6 Load Distribution to Girders

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{3}}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{3}}\right)^{0.1}$$

$$e_g := y_t + h + \frac{t_{se}}{2} \qquad \boxed{e_g = 33.45} \qquad \text{in}$$

LRFD [Eq 4.6.2.2.1-1]

$$K_g := n \cdot \left(I_g + A_g \cdot e_g^{2}\right) \qquad \boxed{K_g = 1868972} \qquad \text{in}^4$$

Criteria for using distribtion factors - Range of Applicability per **LRFD [Table 4.6.2.2.2b-1]**.

$$DeckSpan := \begin{cases} \text{"OK"} & \text{if } 3.5 \le S \le 16 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$DeckThickness := \begin{cases} \text{"OK"} & \text{if } 4.5 \le t_s \le 12 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$BridgeSpan := \begin{cases} \text{"OK"} & \text{if } 20 \le L \le 240 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$NoBeams := \begin{cases} \text{"OK"} & \text{if } ng \ge 4 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$LongitStiffness := \begin{cases} \text{"OK"} & \text{if } 10000 \le K_g \le 7000000 \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$$x := \begin{pmatrix} S & DeckSpan \\ t_s & DeckThickness \\ L & BridgeSpan \\ ng & NoBeams \\ K_g & LongitStiffness \end{pmatrix} \qquad x = \begin{pmatrix} 7.5 & \text{"OK"} \\ 8.0 & \text{"OK"} \\ 130.0 & \text{"OK"} \\ 6.0 & \text{"OK"} \\ 1868972.4 & \text{"OK"} \end{pmatrix}$$

E45-3.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\,3}}\right)^{0.1} \qquad \boxed{g_{i1} = 0.427}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\,3}}\right)^{0.1} \qquad \boxed{g_{i2} = 0.619}$$

$$g_i := \max(g_{i1}, g_{i2}) \qquad \boxed{g_i = 0.619}$$

Note: The distribution factors above already have a multiple lane factor included. For the Wis-SPV Design Check, the distribution factor for One Lane Loaded should be used and the 1.2 multiple presence factor should be divided out.

E45-3.8 Dead Load Moments

The unfactored dead load moments are listed below (values are in kip-ft):

Unfactored Dead Load Interior Girder Moments, (ft-kips)			
Tenth Point	DC non-composite	DC composite	DW composite
0.5	3548	137	141
0.6	3402	99	102
0.7	2970	39	40
0.8	2254	-43	-45
0.9	1253	-147	-151
1.0	0	-272	-281

The DC_{nc} values are the component non-composite dead loads and include the weight of the girder, haunch, diaphragms and the deck.

The DC_c values are the component composite dead loads and include the weight of the parapets.

The DW_c values are the composite dead loads from the future wearing surface.

Note that the girder dead load moments (a portion of DC_{nc}) are calculated based on the CL bearing to CL bearing length. The other DC_{nc} moments are calculated based on the span length (center of bearing at the abutment to centerline of the pier).

E45-3.9 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the impact factor is applied only to the truck portion of the HL-93 loads. A separate analysis run will be required if results without impact are desired.

Unfactored Live Load + Impact Moments per Lane (kip-ft)		
Tenth Point	Truck Pair	Truck + Lane
0.5	--	-921
0.6	--	-1106
0.7	--	-1290
0.8	-1524	-1474
0.9	-2046	-1845
1	-3318	-2517

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$g_i = 0.619$

$M_{LL} = g_i \cdot -3317.97$ $\boxed{M_{LL} = -2055}$ kip-ft

E45-3.10 Composite Girder Section Properties

Calculate the effective flange width in accordance with Chapter 17.2.11.

The effective flange width is calculated as the minimum of the following two values:

$w_e := S \cdot 12$ $\boxed{w_e = 90.00}$ in

The effective width, w_e, must be adjusted by the modular ratio, $n = 1.54$, to convert to the same concrete material (modulus) as the girder.

$w_{eadj} := \dfrac{w_e}{n}$ $\boxed{w_{eadj} = 58.46}$ in

Calculate the composite girder section properties:

effective slab thickness; $t_{se} = 7.50$ in

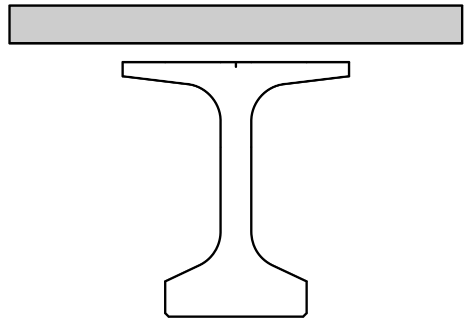
effective slab width; $w_{eadj} = 58.46$ in

haunch thickness; $h = 2.0$ in

total height; $h_c := ht + h + t_{se}$

$h_c = 63.50$ in

$n = 1.540$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	59.75	438	26197	1565294	2055	1567349
Girder	26.3	798	20987	551969	321049	873018
Haunch	55	0	0	0	0	0
Summation		1236	47185			2440367

$\Sigma A := 1236$ in^2

$\Sigma AY := 47185$ in^4

$\Sigma IplusAYsq := 2440367$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$$ $y_{cgb} = -38.2$ in

$$y_{cgt} := ht + y_{cgb}$$ $y_{cgt} = 15.8$ in

$$A_{cg} := \Sigma A$$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^2$$ $I_{cg} = 639053$ in^4

Deck:

$$S_c := n \cdot \frac{I_{cg}}{y_{cgt} + h + t_{se}}$$ $S_c = 38851$ in^4

E45-3.11 Flexural Strength Capacity at Pier

All of the continuity reinforcement is placed in the top mat. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

$bar_{trans} := 5$ (transverse bar size)

$Bar_D(bar_{trans}) = 0.625$ in (transverse bar diameter)

$Bar_{No} = 10$

$Bar_D(Bar_{No}) = 1.27$ in (Assumed bar size)

$$d_e := ht + h + t_s - cover - Bar_D(bar_{trans}) - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_e = 60.24} \text{ in}$$

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

The continuity reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches.

From E19-2, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement is placed at 1/2 of this bar spacing, .

#10 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 3.57}$ in^2/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12} \qquad \boxed{As = 26.80} \text{ in}^2$$

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

$\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$$a := \frac{As \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c} \qquad \boxed{a = 7.883} \text{ in}$$

This is approximately equal to the thickness of the bottom flange height of 7.5 inches.

$$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12} \qquad \boxed{M_n = 7544} \text{ kip-ft}$$

$$M_r := \phi_f \cdot M_n \qquad \boxed{M_r = 6790} \text{ kip-ft}$$

E45-3.12 Design Load Rating

This design example illustrates the rating checks required at the location of maximum negative moment. These checks are also required at the locations of continuity bar cut offs but are not shown here.

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Load Factors taken from Table 45.3-1

$\gamma_{L\_inv} := 1.75$ $\gamma_{DC} := 1.25$ $\gamma_{servLL} := 0.8$ $\phi_c := 1.0$ $\phi_s := 1.0$

$\gamma_{L\_op} := 1.35$ $\gamma_{DW} := 1.50$ $\phi := 0.9$ for flexure

For Flexure

$\boxed{M_n = 7544}$ kip-ft $\boxed{M_{DCc} = 272}$ kip-ft $\boxed{M_{LL} = 2055}$ kip-ft

Inventory Level

$$RF_{Mom\_Inv} := \frac{(\phi_c)(\phi_s)(\phi)(M_n) - \gamma_{DC}\cdot(M_{DCc})}{\gamma_{L\_inv}\cdot(M_{LL})} \qquad \boxed{RF_{Mom\_Inv} = 1.793}$$

Operating Level

$$RF_{Mom\_Op} := \frac{(\phi_c)(\phi_s)(\phi)(M_n) - \gamma_{DC}\cdot(M_{DCc})}{\gamma_{L\_op}\cdot(M_{LL})} \qquad \boxed{RF_{Mom\_Op} = 2.325}$$

E45-3.13 Permit Load Rating

Check the Wisconsin Standard Permit Vehicle per 45.12

For a symetric 130' two span structure:

$MSPV_{LL} := 2738$ kip-ft per lane (includes Dynamic Load Allowance of 33%)

Per 45.12, for the Wisconsin Standard Permit Vehicle (Wis-SPV) Design Check use single lane distribution factor assuming a single trip permit vehicle with no escort vehicles and assuming full dynamic load allowance. Also, divide out the 1.2 multiple presence factor per **MBE [6A.4.5.4.2]** for the single lane distribution factor only.

Single Lane Distribution

$$g_1 := g_{i1}\frac{1}{1.2} \qquad \boxed{g_1 = 0.356}$$

$$M_{SPVLLIM} := \left(MSPV_{LL} + M_{Lane}\right)\cdot g_1 \qquad \boxed{M_{SPVLLIM} = 975} \text{ kip-ft}$$

$$RF_{SPV\_m1} := \frac{\left[(\phi_c)(\phi_s)(\phi)(M_n)\right] - 1.25\cdot(M_{DCc}) - 1.5(M_{DWc})}{1.2(M_{SPVLLIM})} \qquad \boxed{RF_{SPV\_m1} = 5.151}$$

$$Wt_1 := RF_{SPV\_m1}\cdot 190 \qquad \boxed{Wt_1 = 979} \text{ kips >> 190 kips, OK}$$

The rating for the Wis-SPV vehicle is now checked without the Future Wearing Surface. This value is reported on the plans.

$$RF_{SPV\_m\_pln} := \frac{\left[(\phi_c)(\phi_s)(\phi)(M_n)\right] - 1.25\cdot(M_{DCc})}{1.2(M_{SPVLLIM})} \qquad \boxed{RF_{SPV\_m\_pln} = 5.511}$$

$$Wt_{pln} := RF_{SPV\_m\_pln}\cdot 190 \qquad \boxed{Wt_{pln} = 1047} \text{ kips}$$

Since this value is greater than 250 kips, 250 kips is reported on the plans and on the Bridge Load Rating Summary form for the single-lane Permit Load Rating.

Multi-Lane Distribution

$$g_2 := g_{i2} \qquad \boxed{g_2 = 0.619}$$

$$M_{SPVLLIM} := MSPV_{LL}\cdot g_2 \qquad \boxed{M_{SPVLLIM} = 1696} \text{ kip-ft}$$

$$RF_{SPV\_m2} := \frac{\left[(\phi_c)(\phi_s)(\phi)(M_n)\right] - 1.25\cdot(M_{DCc})}{1.3(M_{SPVLLIM})} \qquad \boxed{RF_{SPV\_m2} = 2.925}$$

$$Wt_2 := RF_{SPV\_m2}\cdot 190 \qquad \boxed{Wt_2 = 556} \text{ kips}$$

Since this value is greater than 250 kips, 250 kips is reported on the Bridge Load Rating Summary form for the multi-lane Permit Load Rating.

E45-3.14 Summary of Rating Factors

Interior Girder						
Limit State		Design Load Rating		Legal Load	Permit Load Rating (kips)	
		Inventory	Operating	Rating	Single Lane	Multi-Lane
Strength 1	Flexure	1.79	2.32	N/A	250	250

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E45-4 Steel Girder Rating Example - LRFR

This example shows rating calculations conforming to the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* as supplemented by the *WisDOT Bridge Manual (July 2008)*. This example will rate the design example E24-1 contained in the WisDOT Bridge Manual. **(Note: Example has not been updated for example E24-1 January 2016 updates)**

E45-4.1 Preliminary Data

An interior plate girder will be rated for this example. The girder was designed to be composite throughout. There is no overburden on the structure. In addition, inspection reports reveal no loss of section to any of the main load carrying members.

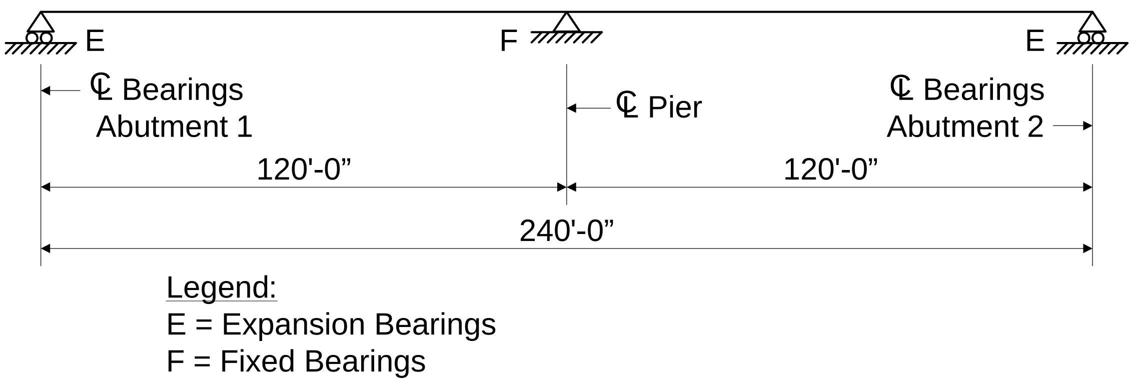

Figure E45-4.1-1
Span Configuration

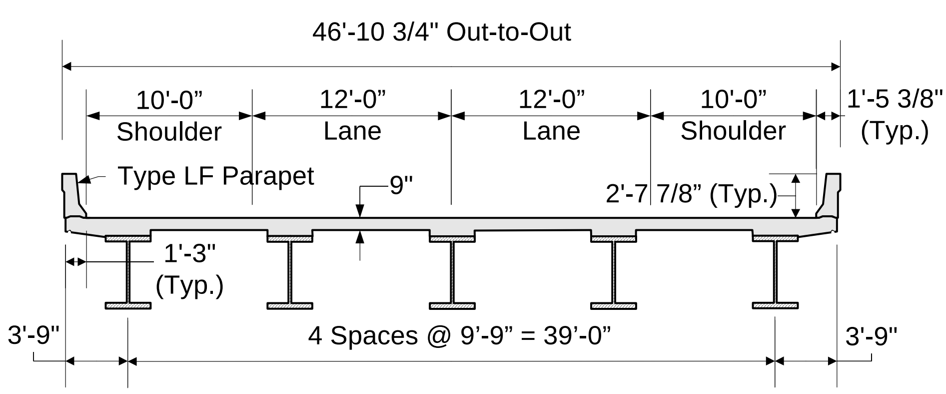

Figure E45-4.1-2
Superstructure Cross Section

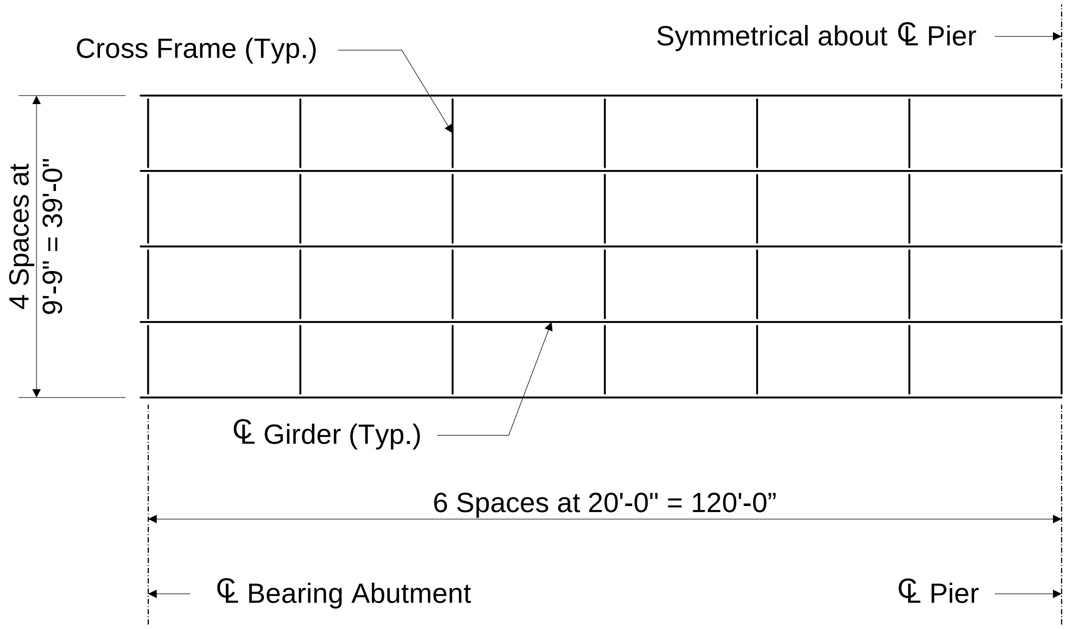

Figure E45-4.1-3
Framing Plan

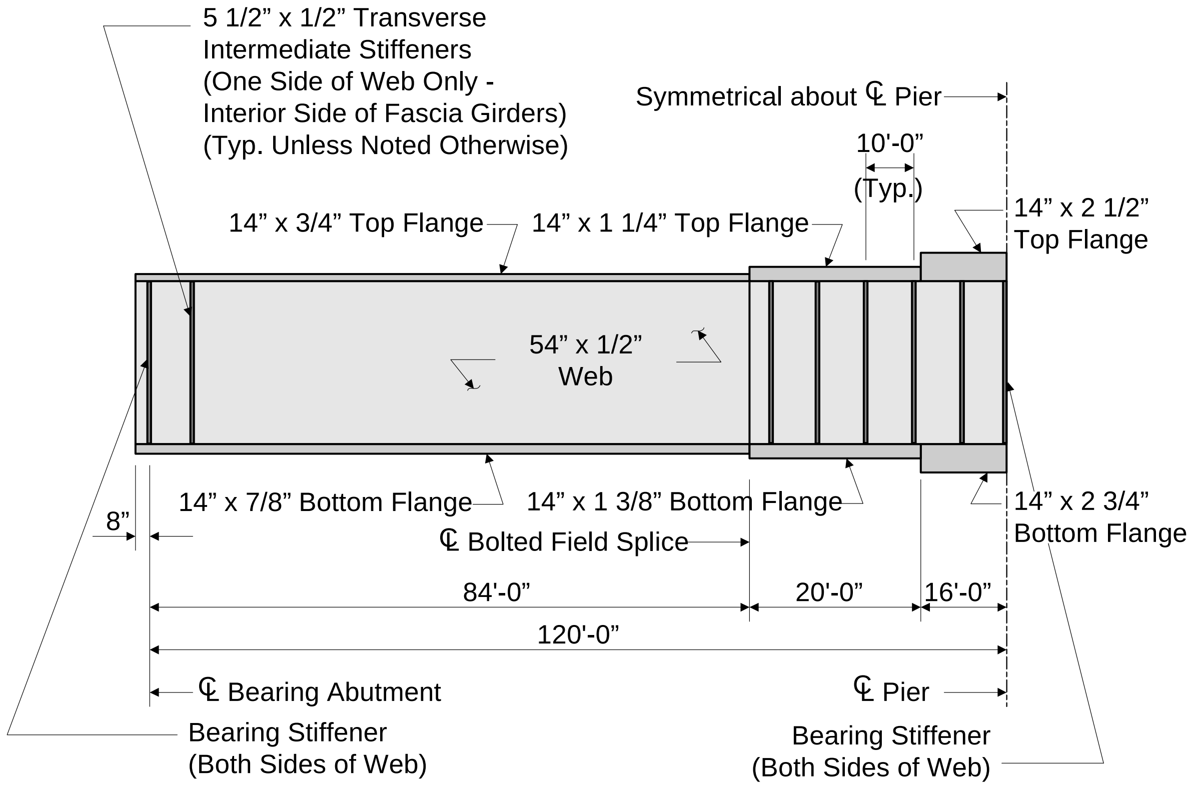

Figure E45-4.1-4
Interior Plate Girder Elevation

$N_{spans} := 2$	Number of spans	
$L := 120$	ft	span length
$Skew := 0$	deg	skew angle
$N_b := 5$	number of girders	
$S := 9.75$	ft	girder spacing
$S_{overhang} := 3.75$	ft	deck overhang
$L_b := 240$	in	cross-frame spacing
$F_{yw} := 50$	ksi	web yield strength
$F_{yf} := 50$	ksi	flange yield strength
$f'_c := 4.0$	ksi	concrete 28-day compressive strength
$f_y := 60$	ksi	reinforcement strength
$E_s := 29000$	ksi	modulus of elasticity
$t_{deck} := 9.0$	in	total deck thickness
$t_s := 8.5$	in	effective deck thickness when 1/2" future wearing surface is removed from total deck thickness
$w_s := 0.490$	kcf	steel density **LRFD[Table 3.5.1-1]**
$w_c := 0.150$	kcf	concrete density **LRFD[Table 3.5.1-1 & C3.5.1]**
$w_{misc} := 0.030$	kip/ft	additional miscellaneous dead load (per girder) per 17.2.4.1
$w_{par} := 0.387$	kip/ft	parapet weight (each)
$w_{fws} := 0.00$	kcf	future wearing surface is not used in rating analysis
$w_{deck} := 46.5$	ft	deck width
$w_{roadway} := 44.0$	ft	roadway width
$d_{haunch} := 3.75$	in	haunch depth (from top of web for design) (for construction, the haunch is measured from the top of the top flange)

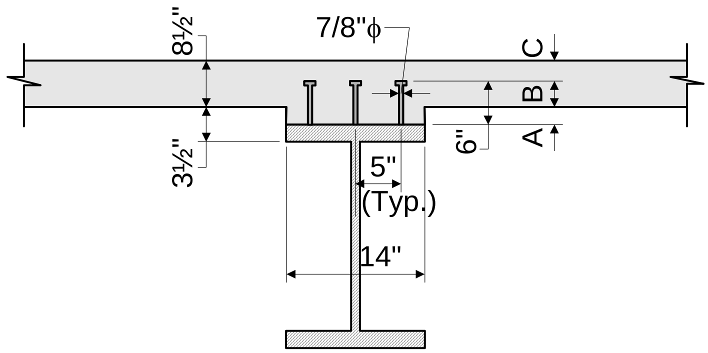

Figure E45-4.1-5
Composite Cross Section at Location of Maximum Positive Moment (0.4L)
(Note: 1/2" Intergral Wearing Surface has been removed for structural calcs.)

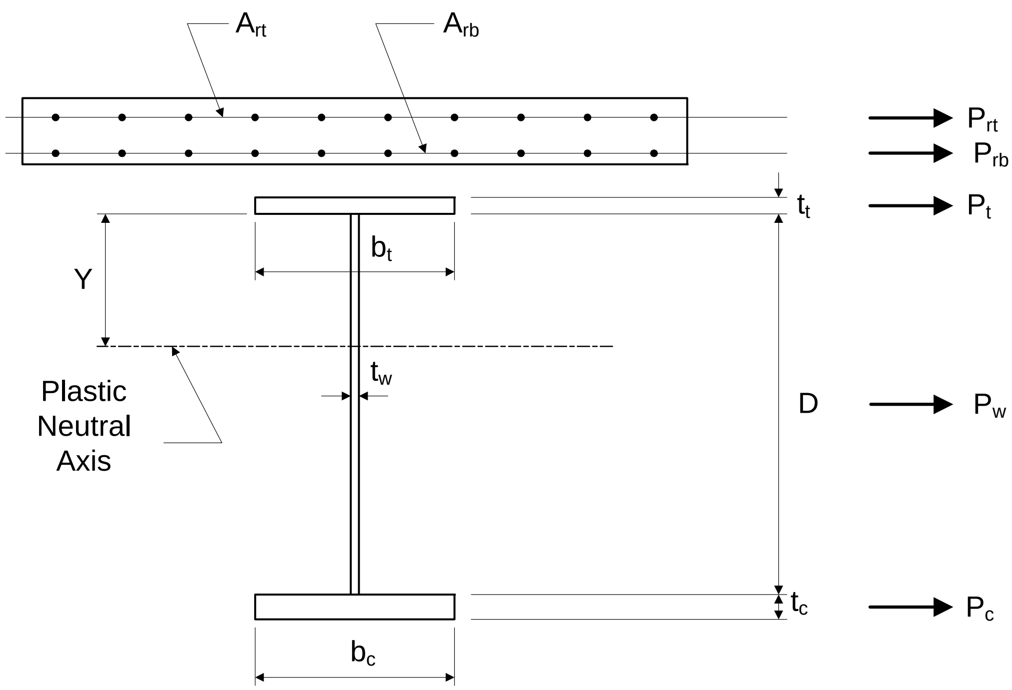

Figure E45-4.1-6
Composite Cross Section at Location of Maximum Negative Moment over Pier

$D := 54$ in

$t_w := 0.5$ in

E45-4.2 Compute Section Properties

Since the superstructure is composite, several sets of section properties must be computed **LRFD [6.10.1.1]**. The initial dead loads (or the noncomposite dead loads) are applied to the girder-only section. For permanent loads assumed to be applied to the long-term composite section, the long-term modular ratio of 3n is used to transform the concrete deck area **LRFD [6.10.1.1.1b]**. For transient loads assumed applied to the short-term composite section, the short-term modular ratio of n is used to transform the concrete deck area.

The modular ratio, n, is computed as follows:

$$n = \frac{E_s}{E_c}$$

Where:

E_s = Modulus of elasticity of steel (ksi)

E_c = Modulus of elasticity of concrete (ksi)

$E_s = 29000.00$ ksi **LRFD [6.4.1]**

$$E_c = 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$$ **LRFD [C5.4.2.4]**

Where:

K_1 = Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction

w_c = Unit weight of concrete (kcf)

f'_c = Specified compressive strength of concrete (ksi)

$w_c = 0.15$ kcf **LRFD [Table 3.5.1-1 & C3.5.1]**

$f'_c = 4.00$ ksi **LRFD [Table 5.4.2.1-1 & 5.4.2.1]**

$K_1 := 1.0$ **LRFD [5.4.2.4]**

$$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$$ $\boxed{E_c = 3834}$ ksi

$$n := \frac{E_s}{E_c}$$ $\boxed{n = 7.6}$ **LRFD [6.10.1.1.1b]**

Therefore, use: $n := 8$

The effective flange width is computed as follows .

For interior beams, the effective flange width is calculated as per **LRFD [4.6.2.6]**:

1. 12.0 times the average thickness of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder:

$$b_{eff2} := \frac{12 \cdot t_s + \frac{14}{2}}{12}$$

This is no longer a valid criteria, however it has been left in place to avoid changing the entire example at this time.

$\boxed{b_{eff2} = 9.08}$ ft

2. The average spacing of adjacent beams:

$$b_{eff3} := S$$

$\boxed{b_{eff3} = 9.75}$ ft

Therefore, the effective flange width is:

$$b_{effflange} := \min\left(b_{eff2}, b_{eff3}\right)$$

$\boxed{b_{effflange} = 9.08}$ ft

or

$\boxed{b_{effflange} \cdot 12 = 109.00}$ in

For this design example, the slab haunch is 3.5 inches throughout the length of the bridge. That is, the bottom of the slab is located 3.5 inches above the top of the web The area of the haunch is conservatively not considered in the section properties for this example.

Based on the plate sizes shown in Figure E453.1-4, the noncomposite and composite section properties for the positive moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder.

Positive Moment Region Section Properties						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	A*y^2 (Inches^4)	I_{total} (Inches^4)
Girder only:						
Top flange	10.500	55.250	580.1	0.5	8441.1	8441.6
Web	27.000	27.875	752.6	6561.0	25.8	6586.8
Bottom flange	12.250	0.438	5.4	0.8	8576.1	8576.9
Total	49.750	26.897	1338.1	6562.3	17043.0	23605.3
Composite (3n):						
Girder	49.750	26.897	1338.1	23605.3	12293.9	35899.2
Slab	38.604	62.875	2427.2	232.4	15843.4	16075.8
Total	88.354	42.617	3765.3	23837.7	28137.3	51975.0
Composite (n):						
Girder	49.750	26.897	1338.1	23605.3	31511.0	55116.2
Slab	115.813	62.875	7281.7	697.3	13536.3	14233.6
Total	165.563	52.064	8619.8	24302.5	45047.3	69349.8
Section	y_{botgdr} (Inches)	y_{topgdr} (Inches)	$y_{topslab}$ (Inches)	S_{botgdr} (Inches^3)	S_{topgdr} (Inches^3)	$S_{topslab}$ (Inches^3)
Girder only	26.897	28.728	---	877.6	821.7	---
Composite (3n)	42.617	13.008	24.508	1219.6	3995.5	2120.7
Composite (n)	52.064	3.561	15.061	1332.0	19474.0	4604.5

Table E45-4.2-1

Positive Moment Region Section Properties

Similarly, the noncomposite and composite section properties for the negative moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder **LRFD [6.6.1.2.1, 6.10.5.1, 6.10.4.2.1]**.

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not. However, per 45.6.3, only the top longitudinal mat of steel is used for rating purposes. Per the design example, the amount of longitudinal steel within the effective slab area is 6.39 in^2. This number will be used for the calculations below.

Negative Moment Region Section Properties						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	A*y^2 (Inches^4)	I_{total} (Inches^4)
Girder only:						
Top flange	35.000	58.000	2030.0	18.2	30009.7	30027.9
Web	27.000	29.750	803.3	6561.0	28.7	6589.7
Bottom flange	38.500	1.375	52.9	24.3	28784.7	28809.0
Total	100.500	28.718	2886.2	6603.5	58823.1	65426.6
Composite (deck concrete using 3n):						
Girder	100.500	28.718	2886.2	65426.6	10049.0	75475.6
Slab	38.604	64.750	2499.6	232.4	26161.1	26393.5
Total	139.104	38.718	5385.8	65659.0	36210.1	101869.2
Composite (deck concrete using n):						
Girder	100.500	28.718	2886.2	65426.6	37401.0	102827.7
Slab	115.813	64.750	7498.9	697.3	32455.9	33153.2
Total	216.313	48.009	10385.0	66123.9	69857.0	135980.9
Composite (deck reinforcement only):						
Girder	100.500	28.718	2886.2	65426.6	466.3	65892.9
Deck reinf.	6.390	64.750	413.8	0.0	7333.8	7333.8
Total	106.890	30.872	3299.9	65426.6	7800.1	73226.7
Section	y_{botgdr} (Inches)	y_{topgdr} (Inches)	y_{deck} (Inches)	S_{botgdr} (Inches^3)	S_{topgdr} (Inches^3)	S_{deck} (Inches^3)
Girder only	28.718	30.532	---	2278.2	2142.9	---
Composite (3n)	38.718	20.532	30.282	2631.1	4961.4	3364.0
Composite (n)	48.009	11.241	20.991	2832.4	12097.4	6478.2
Composite (rebar)	30.872	28.378	33.878	2371.9	2580.4	2161.5

Table E45-4.2-3
Negative Moment Region Section Properties

E45-4.3 Dead Load Analysis - Interior Girder

Dead Load Components		
Resisted by	Type of Load Factor	
	DC	DW
Noncomposite section	• Steel girder • Concrete deck • Concrete haunch • Stay-in-place deck forms • Misc. (including cross-frames, stiffeners, etc.)	
Composite section	• Concrete parapets	• Future wearing surface & utilities

Table E45-4.3-1
Dead Load Components

COMPONENTS AND ATTACHMENTS: DC1 (NON-COMPOSITE)

GIRDER:

For the steel girder, the dead load per unit length varies due to the change in plate sizes. The moments and shears due to the weight of the steel girder can be computed using readily available analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

DECK:

For the concrete deck, the dead load per unit length for an interior girder is computed as follows:

$w_c = 0.150$ kcf

$S = 9.75$ ft

$t_{deck} = 9.00$ in

$$DL_{deck} := w_c \cdot S \cdot \frac{t_{deck}}{12} \qquad \boxed{DL_{deck} = 1.097} \quad \text{kip/ft}$$

HAUNCH:

For the concrete haunch, the dead load per unit length varies due to the change in top flange plate sizes. The moments and shears due to the weight of the concrete haunch can be computed using readily available analysis software. Since the top flange plate sizes are entered as input, the moments and shears due to the concrete haunch are computed based on the actual, varying haunch thickness.

MISC:

For the miscellaneous dead load (including cross-frames, stiffeners, and other miscellaneous structural steel), the dead load per unit length is assumed to be as follows (17.2.4.1):

$DL_{misc} := 0.030$ kip/ft

COMPONENTS AND ATTACHMENTS: DC2 (COMPOSITE)

PARAPET:

For the concrete parapets, the dead load per unit length is computed as follows, assuming that the superimposed dead load of the two parapets is distributed uniformly among all of the girders **LRFD [4.6.2.2.1]**:

$w_{par} = 0.39$ kip/ft

$N_b = 5$

$DL_{par} := \frac{w_{par} \cdot 2}{N_b}$ $\boxed{DL_{par} = 0.155}$ kip/ft

WEARING SURFACE: DW (COMPOSITE)

FUTURE WEARING SURFACE:

For this example, future wearing surface is only applied for permit vehicle rating checks.

Since the plate girder and its section properties are not uniform over the entire length of the bridge, an analysis software was used to compute the dead load moments and shears.

The following two tables present the unfactored dead load moments and shears, as computed by an analysis computer program. Since the bridge is symmetrical, the moments and shears in Span 2 are symmetrical to those in Span 1.

Dead Load Moments (Kip-feet)											
Dead Load Component	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Steel girder	0.0	71.8	119.3	142.5	141.3	115.8	66.0	-8.2	-110.2	-244.4	-423.9
Concrete deck & haunches	0.0	480.5	796.7	948.6	936.1	759.4	418.4	-86.9	-756.0	-1588.1	-2581.3
Miscellaneous Steel Weight	0.0	12.6	21.0	25.0	24.6	20.0	11.0	-2.3	-19.9	-41.8	-68.0
Concrete parapets	0.0	67.7	113.1	136.1	136.9	115.3	71.4	5.1	-83.4	-194.3	-327.5
Future wearing surface	0.0	76.9	128.4	154.6	155.4	130.9	81.0	5.8	-94.7	-220.6	-371.9

Table 45E-4.3-2
Dead Load Moments

Dead Load Shears (Kips)											
Dead Load Component	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Steel girder	7.0	5.0	2.9	0.9	-1.1	-3.1	-5.2	-7.2	-9.8	-12.9	-17.0
Concrete deck & haunches	46.9	33.2	19.5	5.8	-7.9	-21.6	-35.3	-49.0	-62.6	-76.1	-89.5
Miscellaneous Steel Weight	1.2	0.9	0.5	0.2	-0.2	-0.6	-0.9	-1.3	-1.7	-2.0	-2.4
Concrete parapets	6.6	4.7	2.9	1.0	-0.9	-2.7	-4.6	-6.5	-8.3	-10.2	-12.0
Future wearing surface	7.5	5.4	3.2	1.1	-1.0	-3.1	-5.2	-7.3	-9.4	-11.6	-13.7

Table 45E-4.3-3
Dead Load Shears

E45-4.4 Compute Live Load Distribution Factors for Interior Girder

The live load distribution factors for an interior girder are computed as follows **LRFD [4.6.2.2.2]**:

First, the longitudinal stiffness parameter, K_g, must be computed **LRFD [4.6.2.2.1]**:

$$K_g := n \cdot \left(I + A \cdot e_g^{\,2} \right)$$

Where:

I	= Moment of inertia of beam (in^4)
A	= Area of stringer, beam, or girder (in^2)
e_g	= Distance between the centers of gravity of the basic beam and deck (in)

Longitudinal Stiffness Parameter, K_g				
	Region A (Pos. Mom.)	Region B (Intermediate)	Region C (At Pier)	Weighted Average *
Length (Feet)	84	20	16	
n	8	8	8	
I ($Inches^4$)	23,605.3	34,639.8	65,426.6	
A ($Inches^2$)	49.750	63.750	100.500	
e_g (Inches)	35.978	35.777	36.032	
K_g ($Inches^4$)	704,020	929,915	1,567,250	856,767

Table E45-4.4-1
Longitudinal Stiffness Parameter

After the longitudinal stiffness parameter is computed, **LRFD [Table 4.6.2.2.1-1]** is used to find the letter corresponding with the superstructure cross section. The letter corresponding with the superstructure cross section in this design example is "a."

If the superstructure cross section does not correspond with any of the cross sections illustrated in **LRFD [Table 4.6.2.2.1-1]**, then the bridge should be analyzed as presented in **LRFD [4.6.3]**.

Based on cross section "a", **LRFD [Table 4.6.2.2.2b-1 & Table 4.6.2.2.2.3a-1]** are used to compute the distribution factors for moment and shear, respectively.

For the 0.4L point:

$K_g = 856766.65$ in^4

$L := 120$ ft

For one design lane loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [4.6.2.2.2b-1]**:

$$g_{m1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0L \cdot t_s^{\,3}}\right)^{0.1}$$

$\boxed{g_{m1} = 0.466}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [Table 4.6.2.2.2b-1]**:

$$g_{m2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0 \cdot L \cdot t_s^{\,3}}\right)^{0.1}$$

$\boxed{g_{m2} = 0.688}$ lanes

The live load distribution factors for shear for an interior girder are computed in a similar manner. The range of applicability is similar to that for moment **LRFD [Table 4.6.2.2.3a-1]**.

For one design lane loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{v1} := 0.36 + \frac{S}{25.0}$$

$\boxed{g_{v1} = 0.750}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{v2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$\boxed{g_{v2} = 0.935}$ lanes

Since this bridge has no skew, the skew correction factor does not need to be considered for this design example **LRFD [4.6.2.2.2e & 4.6.2.2.3c]**.

HL-93 Live Load Effects (for Interior Beams)											
	Location in Span 1										
Live Load Effect	**0.0L**	**0.1L**	**0.2L**	**0.3L**	**0.4L**	**0.5L**	**0.6L**	**0.7L**	**0.8L**	**0.9L**	**1.0L**
Maximum positive moment (K-ft)	0.0	848.1	1435.4	1780.1	1916.6	1859.0	1626.9	1225.6	699.7	263.6	0.0
Maximum negative moment (K-ft)	0.0	-134.3	-268.7	-403.0	-537.4	-671.7	-806.0	-940.4	-1087.0	-1591.6	-2414.2
Maximum positive shear (kips)	111.1	92.9	76.0	60.4	46.4	34.0	23.3	14.5	7.6	3.0	0.0
Maximum negative shear (kips)	-15.2	-15.7	-21.9	-35.0	-49.2	-63.6	-78.1	-92.3	-106.1	-119.3	-132.0

Table 45E-4.4-2
Live Load Effects

The live load values for HL-93 loading, as presented in the previous table, are computed based on the product of the live load effect per lane and live load distribution factor. These values also include the effects of dynamic load allowance. However, it is important to note that the dynamic load allowance is applied only to the design truck or tandem. The dynamic load allowance is not applied to pedestrian loads or to the design lane load **LRFD [3.6.1, 3.6.2, 4.6.2.2]**.

Two sections will be checked for illustrative purposes. First, the ratings will be performed for the location of maximum positive moment, which is at 0.4L in Span 1. Second, the ratings will be performed for the location of maximum negative moment and maximum shear, which is at th pier.

The following are for the location of maximum positive moment, which is at 0.4L in Span 1, as shown in Figure E453.4-1.

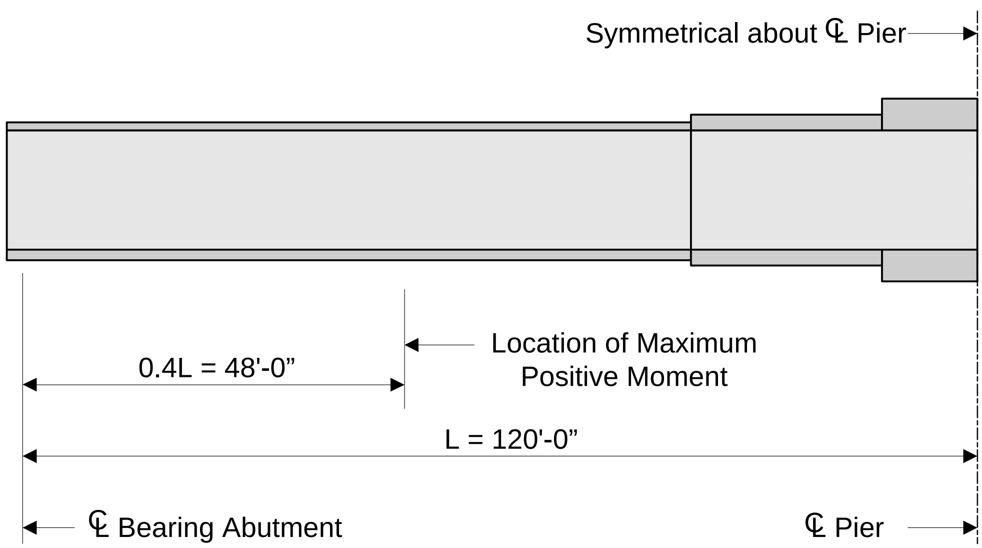

Figure E45-4.4-1
Location of Maximum Positive Moment

E45-4.5 Compute Plastic Moment Capacity - Positive Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**.

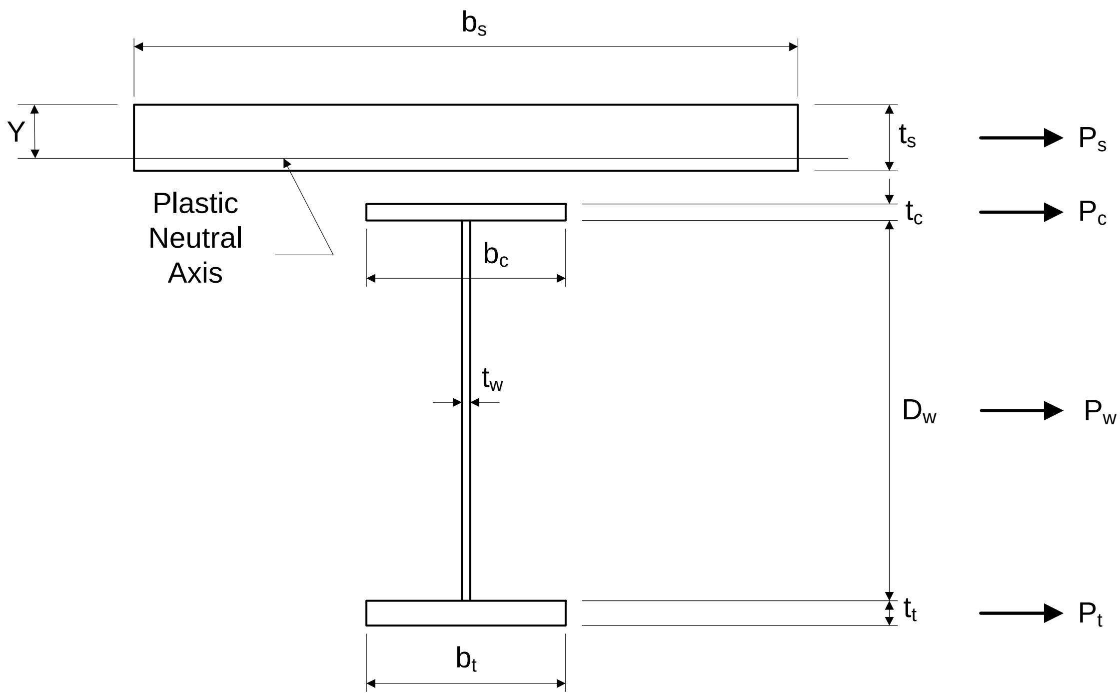

Figure E45-4.5-1
Computation of Plastic Moment Capacity for Positive Bending Sections

For the tension flange:

$$P_t = F_{yt} \cdot b_t \cdot t_t$$

Where:

F_{yt} = Specified minimum yield strength of a tension flange (ksi)

b_t = Full width of the tension flange (in)

t_t = Thickness of tension flange (in)

$F_{yt} := 50$ ksi

$b_t := 14$ in

$t_t := 0.875$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 613}$ kips

For the web:

$P_w := F_{yw} \cdot D \cdot t_w$

Where:

F_{yw} = Specified minimum yield strength of a web (ksi)

$F_{yw} := 50$ ksi

$D = 54.00$ in

$t_w = 0.50$ in

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

For the compression flange:

$P_c = F_{yc} \cdot b_c \cdot t_c$

Where:

F_{yc} = Specified minimum yield strength of a compression flange (ksi)

b_c = Full width of the compression flange (in)

t_c = Thickness of compression flange (in)

$F_{yc} := 50$ ksi

$b_c := 14$ in

$t_c := 0.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 525}$ kips

For the slab:

$P_s = 0.85 \cdot f'_c \cdot b_s \cdot t_s$

Where:

b_s = Effective width of concrete deck (in)

t_s = Thickness of concrete deck (in)

$f'_c = 4.00$ ksi

$b_s := 109$ in

$t_s = 8.50$ in

$P_s := 0.85 \cdot f'_c \cdot b_s \cdot t_s$ $\boxed{P_s = 3150}$ kips

The forces in the longitudinal reinforcement may be conservatively neglected in regions of positive flexure.

Check the location of the plastic neutral axis, as follows:

$P_t + P_w = 1963$ kips $\boxed{P_c + P_s = 3675}$ kips

$P_t + P_w + P_c = 2488$ kips $\boxed{P_s = 3150}$ kips

Therefore, the plastic neutral axis is located within the slab **LRFD [Table D6.1-1]**.

$$Y := (t_s)\cdot\left(\frac{P_c + P_w + P_t}{P_s}\right) \qquad \boxed{Y = 6.71} \text{ in}$$

Check that the position of the plastic neutral axis, as computed above, results in an equilibrium condition in which there is no net axial force.

$$\text{Compression} := 0.85\cdot f'_c\cdot b_s\cdot Y \qquad \boxed{\text{Compression} = 2487} \text{ kips}$$

$$\text{Tension} := P_t + P_w + P_c \qquad \boxed{\text{Tension} = 2488} \text{ kips} \quad \text{OK}$$

The plastic moment, M_p, is computed as follows, where d is the distance from an element force (or element neutral axis) to the plastic neutral axis **LRFD [Table D6.1-1]**:

$$d_c := \frac{-t_c}{2} + 3.75 + t_s - Y \qquad \boxed{d_c = 5.16} \text{ in}$$

$$d_w := \frac{D}{2} + 3.75 + t_s - Y \qquad \boxed{d_w = 32.54} \text{ in}$$

$$d_t := \frac{t_t}{2} + D + 3.75 + t_s - Y \qquad \boxed{d_t = 59.98} \text{ in}$$

$$M_p := \frac{\frac{Y^2\cdot P_s}{2\cdot t_s} + \left(P_c\cdot d_c + P_w\cdot d_w + P_t\cdot d_t\right)}{12} \qquad \boxed{M_p = 7643} \text{ kip-ft}$$

E45-4.6 Determine if Section is Compact or Noncompact - Positive Moment Region

Since the section is in a straight bridge, the next step in the design process is to determine if the section is compact or noncompact. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

If the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the compact-section web slenderness provisions, as follows **LRFD [6.10.6.2.2]**:

$$\frac{2\cdot D_{cp}}{t_w} \le 3.76\cdot\sqrt{\frac{E}{F_{yc}}}$$

Where:

D_{cp} = Depth of web in compression at the plastic moment (in)

Since the plastic neutral axis is located within the slab,

$D_{cp} := 0$ in

Therefore the web is deemed compact. Since this is a composite section in positive flexure and there are no holes in the tension flange at this section, the flexural resistance is computed as defined by the composite compact-section positive flexural resistance provisions of **LRFD [6.10.7.1.2]**.

E45-4.7 Flexural Resistance of Composite Section - Positive Moment Region

Since the section was determined to be compact, and since it is a composite section in the positive moment region with no holes in the tension flange, the flexural resistance is computed in accordance with **LRFD [6.10.7.1.2]**.

$$M_{n\_0.4L} = 1.3 \cdot R_h \cdot M_y$$

Where:

R_h = Hybrid factor

M_y = Yield Moment (kip-in)

All design sections of this girder are homogenous. That is, the same structural steel is used for the top flange, the web, and the bottom flange. Therefore, the hybrid factor, R_h, is as follows **LRFD [6.10.1.10.1]**:

$R_h := 1.0$

The yield moment, M_y, is computed as follows **LRFD [Appendix D6.2.2]**:

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

Where:

M_{D1} = Bending moment caused by the factored permanent load applied before the concrete deck has hardened or is made composite (kip-in)

S_{NC} = Noncomposite elastic section modulus (in^3)

M_{D2} = Bending moment caused by the factored permanent load applied to the long-term composite section (kip-in)

S_{LT} = Long-term composite elastic section modulus (in^3)

M_{AD} = Additional bending moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange (kip-in)

S_{ST} = Short-term composite elastic section modulus (in^3)

$M_y = M_{D1} + M_{D2} + M_{AD}$

$F_y := 50$ ksi

$M_{D1} := \left[1.25 \cdot \left(M_{girder} + M_{deck} + M_{misc}\right)\right]$ $\boxed{M_{D1} = 1378}$ kip-ft

$M_{D2} := \left(1.25 \cdot M_{DC2}\right)$ $\boxed{M_{D2} = 171}$ kip-ft

For the bottom flange:

$S_{NC\_pos} = 877.63$ in³

$S_{LT\_pos} = 1219.60$ in³

$S_{ST\_pos} = 1332.01$ in³

$$M_{AD} := \left[\frac{S_{ST\_pos}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC\_pos}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos}}{12^3}}\right)\right] \quad \boxed{M_{AD} = 3272} \text{ kip-ft}$$

$M_{ybot} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ybot} = 4821}$ kip-ft

For the top flange:

$S_{NC\_pos\_top} = 821.67$ in³

$S_{LT\_pos\_top} = 3995.47$ in³

$S_{ST\_pos\_top} = 19473.97$ in³

$$M_{AD} := \frac{S_{ST\_pos\_top}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC\_pos\_top}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos\_top}}{12^3}}\right)$$

$\boxed{M_{AD} = 47658}$ kip-ft

$M_{ytop} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ytop} = 49207}$ kip-ft

The yield moment, M_y, is the lesser value computed for both flanges. Therefore, M_y is determined as follows **LRFD [Appendix D6.2.2]**:

$M_y := \min\left(M_{ybot}, M_{ytop}\right)$ $\boxed{M_y = 4821}$ kip-ft

Therefore, for the positive moment region of this design example, the nominal flexural resistance is computed as follows **LRFD [6.10.7.1.2]**:

$D_p \le 0.1 D_t$

$D_p := Y$ $\boxed{D_p = 6.71}$ in

$D_t := 0.875 + 54 + .75 + 8$ $\boxed{D_t = 63.63}$ in

$\boxed{0.1 \cdot D_t = 6.36}$ $< D_p$

Therefore

$$M_{n\_0.4L} := M_p \cdot \left(1.07 - 0.7 \cdot \frac{D_p}{D_t}\right)$$ $\boxed{M_{n\_0.4L} = 7614}$ kip-ft

Since this is neither a simple span nor a continuous span where the span and the sections in the negative-flexure region over the interior supports satisfy the special conditions outlined at the end of **LRFD[6.10.7.1.2]**, the nominal flexural resistance of the section must not exceed the following:

$M_{n\_0.4L} := 1.3 \cdot R_h \cdot M_y$ $\boxed{M_{n\_0.4L} = 6267}$ kip-ft

The ductility requirement is checked as follows **LRFD [6.10.7.3]**:

$$D_p \le 0.42 D_t$$

Where:

D_p = Distance from top of the concrete deck to the neutral axis of the composite section at the plastic moment (in)

D_t = Total depth of the composite section (in)

$\boxed{0.42 \cdot D_t = 26.72}$ in OK

The factored flexural resistance, M_r, is computed as follows (note that since there is no curvature, skew and wind load is not considered under the Strength I load combination, the flange lateral bending stress is taken as zero in this case **LRFD [6.10.7.1.1]**:

$$M_u + \frac{1}{3}(0) \le \phi_f M_n$$

Where:

M_u = Moment due to the factored loads (kip-in)

M_n = Nominal flexural resistance of a section (kip-in)

$\phi_f := 1.00$

$M_r := \phi_f \cdot M_{n\_0.4L}$ $\boxed{M_r = 6267}$ kip-ft

E45-4.8 Design Load Rating @ 0.4L

$$RF = \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC}(DC)}{\gamma_L(LLIM)}$$

Where:

Load Factors per Table 45.3-1

$\gamma_{Linv} := 1.75$

$\gamma_{Lop} := 1.35$

$\gamma_{DC} := 1.25$

Resistance Factors

$\phi := 1.0$ **MBE [6A.7.3]**

$\phi_c := 1.0$ per 45.3.7.4

$\phi_s := 1.0$ per 45.3.7.5

$M_{DC1} := M_{girder} + M_{deck} + M_{misc}$ $M_{DC1} = 1102.07$ ft – kips

$M_{LLIM} := M_{LL}$ $M_{LLIM} = 1916.55$ ft – kips

A. Strength Limit State

Inventory

$$RF_{inv\_0.4L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot M_{DC1} - \gamma_{DC} \cdot M_{DC2}}{\gamma_{Linv} \cdot (M_{LLIM})}$$

$RF_{inv\_0.4L} = 1.41$

Operating

$$RF_{op\_0.4L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot M_{DC1} - \gamma_{DC} \cdot M_{DC2}}{\gamma_{Lop} \cdot (M_{LLIM})}$$

$RF_{op\_0.4L} = 1.82$

B. Service II Limit State

$$RF = \frac{f_R - \gamma_D \cdot (f_D)}{\gamma_L \cdot (f_{LLIM})}$$

Allowable Flange Stress per **LRFD 6.10.4.2.2**

$f_R = 0.95 R_b \cdot R_h \cdot F_y$

Checking only the tension flange as compression flanges typically do not control for composite sections.

$R_b := 1.0$ For tension flanges

$R_h := 1.0$ For non-hybrid sections

$f_R := 0.95 \cdot R_b \cdot R_h \cdot F_y$

$f_R = 47.50$ ksi

$f_D = f_{DC1} + f_{DC2}$

$$f_D := \left(\frac{M_{DC1} \cdot 12}{S_{NC\_pos}}\right) + \left(\frac{M_{DC2} \cdot 12}{S_{LT\_pos}}\right)$$

$f_D = 16.42$ ksi

$$f_{LLIM} := \frac{M_{LLIM} \cdot 12}{S_{ST\_pos}}$$

$f_{LLIM} = 17.27$ ksi

Load Factors Per Table 45.3-1

$\gamma_D := 1.0$

$\gamma_{Lin} := 1.3$ Inventory

$\gamma_{Lop} := 1.0$ Operating

Inventory

$$RF_{inv\_0.4L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lin} \cdot f_{LLIM}}$$

$RF_{inv\_0.4L\_service} = 1.38$

Operating

$$RF_{op\_0.4L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lop} \cdot f_{LLIM}}$$

$RF_{op\_0.4L\_service} = 1.80$

E45-4.9 Check Section Proportion Limits - Negative Moment Region

Now the specification checks are repeated for the location of maximum negative moment, whicl is at the pier, as shown in Figure 24E1.17-1. This is also the location of maximum shear in this case.

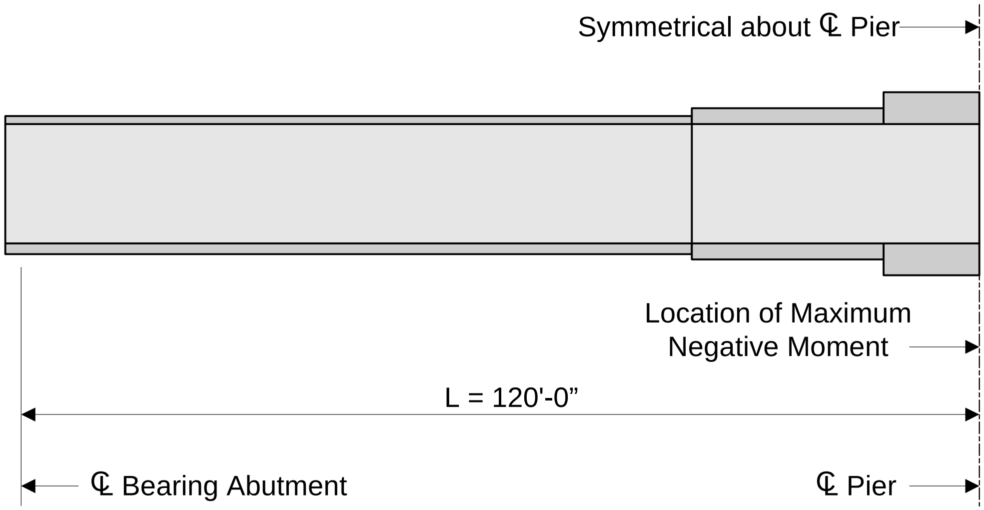

Figure E45-4.9-1
Location of Maximum Negative Moment

Several checks are required to ensure that the proportions of the girder section are within specified limits **LRFD [6.10.2]**.

The first section proportion check relates to the web slenderness **LRFD [6.10.2.1]**. For a section without longitudinal stiffeners, the web must be proportioned such that:

$$\frac{D}{t_w} \le 150 \qquad \boxed{\frac{D}{t_w} = 108.00} \qquad \text{OK}$$

The second set of section proportion checks relate to the general proportions of the section **LRFD [6.10.2.2]**. The compression and tension flanges must be proportioned such that:

$$\frac{b_f}{2 \cdot t_f} \le 12.0$$

$b_f := 14$

$t_f := 2.50$ $\qquad \boxed{\frac{b_f}{2 \cdot t_f} = 2.80}$ OK

$$b_f \ge \frac{D}{6} \qquad \boxed{\frac{D}{6} = 9.00} \quad \text{in} \qquad \text{OK}$$

$$t_f \ge 1.1 \cdot t_w \qquad \boxed{1.1t_w = 0.55} \quad \text{in} \qquad \text{OK}$$

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10$$

$I_{yc} := \dfrac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in⁴

$I_{yt} := \dfrac{2.50 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in⁴

$\boxed{\dfrac{I_{yc}}{I_{yt}} = 1.100}$ OK

E45-4.10 Compute Plastic Moment Capacity - Negative Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**. For composite sections in negative flexure, the concrete deck is ignored and the longitudinal deck reinforcement is included in the computation of M_p.

The plastic force in the tension flange, P_t, is calculated as follows:

$t_t := 2.50$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 1750}$ kips

The plastic force in the web, P_w, is calculated as follows:

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

The plastic force in the compression flange, P_c, is calculated as follows:

$t_c := 2.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 1925}$ kips

The plastic force in the top layer of longitudinal deck reinforcement, P_{rt}, used to compute the plastic moment is calculated as follows:

$$P_{rt} = F_{yrt} \cdot A_{rt}$$

Where:

F_{yrt} = Specified minimum yield strength of the top layer of longitudinal concrete deck reinforcement (ksi)

A_{rt} = Area of the top layer of longitudinal reinforcement within the effective concrete deck width (in²)

$F_{yrt} := 60$ ksi

$$A_{rt} := 0.44 \cdot \left(\frac{b_{effflange} \cdot 12}{7.5} \right)$$ $\boxed{A_{rt} = 6.39}$ in^2

$$P_{rt} := F_{yrt} \cdot A_{rt}$$ $\boxed{P_{rt} = 384}$ kips

The plastic force in the bottom layer of longitudinal deck reinforcement, P_{rb}, used to compute the plastic moment is calculated as follows (WisDOT Policy is to ignore bottom mat steel)

$$P_{rb} = F_{yrb} \cdot A_{rb}$$

Where:

F_{yrb} = Specified minimum yield strength of the bottom layer of longitudinal concrete deck reinforcement (ksi)

A_{rb} = Area of the bottom layer of longitudinal reinforcement within the effective concrete deck width (in^2)

$F_{yrb} := 60$ ksi

$$A_{rb} := 0 \cdot \left(\frac{b_{effflange} \cdot 12}{1} \right)$$ $\boxed{A_{rb} = 0.00}$ in^2

$$P_{rb} := A_{rb} \cdot F_{yrb}$$ $\boxed{P_{rb} = 0}$ kips

NOTE: For continuous girder type bridges, the negative moment steel shall conservatively consist of only the top mat of steel over the piers per **45.6.3**

Check the location of the plastic neutral axis, as follows:

$\boxed{P_c + P_w = 3275}$ kips

$\boxed{P_t + P_{rb} + P_{rt} = 2134}$ kips

$\boxed{P_c + P_w + P_t = 5025}$ kips

$\boxed{P_{rb} + P_{rt} = 384}$ kips

Therefore the plastic neutral axis is located within the web **LRFD [Appendix Table D6.1-2]**.

$$Y := \left(\frac{D}{2} \right) \cdot \left(\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1 \right)$$ $\boxed{Y = 22.83}$ in

Although it will be shown in the next design step that this section qualifies as a nonslender web section at the strength limit state, the optional provisions of Appendix A to **LRFD [6]** are not employed in this example. Thus, the plastic moment is not used to compute the flexural resistance and therefore does not need to be computed.

E45-4.11 Determine if Section is a Compact-Web, Noncompact-Web, or Slender-Web Section - Negative Moment Region

Since the section is in a straight bridge, the next step is to determine if the section is a compact-web, noncompact-web, or slender-web section. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

Where the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the noncompact-web slenderness limit, as follows **LRFD [6.10.6.2.3]**:

$$\frac{2 \cdot D_c}{t_w} \le 5.7 \cdot \sqrt{\frac{E}{F_{yc}}}$$

At sections in negative flexure, D_c of the composite section consisting of the steel section plus the longitudinal reinforcement is to be used at the strength limit state.

$D_c := 30.872 - 2.75$ (see Figure 24E1.2-1 and Table 24E1.3-2)

$\boxed{D_c = 28.12}$ in

$$\boxed{\frac{2 \cdot D_c}{t_w} = 112.5}$$

$$\boxed{5.7 \cdot \sqrt{\frac{E_s}{F_{yc}}} = 137.3}$$

The section is a nonslender web section (i.e. either a compact-web or noncompact-web section). Next, check:

$I_{yc} := \dfrac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in^4

$I_{yt} := \dfrac{2.5 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in^4

$\boxed{\dfrac{I_{yc}}{I_{yt}} = 1.10}$ > 0.3 OK

Therefore, the web qualifies to use the optional provisions of **LRFD [Appendix A6]** to compute the flexural resistance. However, since the web slenderness is closer to the noncompact web slenderness limit than the compact web slenderness limit in this case, the simpler equations of **LRFD [6.10.8]**, which assume slender-web behavior and limit the resistance to Fyc or below, will conservatively be applied in this example to compute the flexural resistance at the strength limit state. The investigation proceeds by calculating the flexural resistance of the discretely braced compression flange.

E45-4.12 Rating for Flexure - Strength Limit State - Negative Moment Region

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance **LRFD [6.10.8.2.2 & 6.10.8.2.3]**.

Local buckling resistance **LRFD [6.10.8.2.2]**:

$b_{fc} := 14$ (see Figure 24E1.2-1)

$t_{fc} := 2.75$ (see Figure 24E1.2-1)

$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}}$ $\boxed{\lambda_f = 2.55}$

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_{yc}}}$ $\boxed{\lambda_{pf} = 9.15}$

Since $\lambda_f < \lambda_{pf}$, F_{nc} is calculated using the following equation:

$F_{nc} := R_b \cdot R_h \cdot F_{yc}$

Since $2D_c/t_w$ is less than λ_{rw} (calculated above), R_b is taken as 1.0 **LRFD [6.10.1.10.2]**.

$\boxed{F_{nc} = 50.00}$ ksi

Lateral torsional buckling resistance **LRFD [6.10.8.2.3]**:

$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}}$ $\boxed{r_t = 3.82}$ in

$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}}$ $\boxed{L_p = 91.90}$ in

$F_{yr} := \max\left(\min\left(0.7 \cdot F_{yc}, F_{yw}\right), 0.5 \cdot F_{yc}\right)$ $\boxed{F_{yr} = 35.00}$ ksi

$L_r := \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}}$ $\boxed{L_r = 345.07}$ in

$L_b = 240.00$

The moment gradient correction factor, C_b, is computed as follows:

Where the variation in the moment along the entire length between brace points is concave in shape, which is the case here, $f_1 = f_0$. (calculated below based on the definition of f_0 given in **LRFD [6.10.8.2.3]**).

$M_{NCDC0.8L} := 110.2 + 756.0 + 19.9$ $\boxed{M_{NCDC0.8L} = 886.10}$ kip-ft

$S_{NCDC0.8L} := 2278.2$ in³

$M_{par0.8L} := 83.4$ kip-ft

$M_{LL0.8L} := 1087.0$ kip-ft

$S_{rebar0.8L} := 2371.9$ in³

$$f_1 := 1.25 \cdot \frac{M_{NCDC0.8L} \cdot 12}{S_{NCDC0.8L}} + 1.25 \cdot \frac{M_{par0.8L} \cdot 12}{S_{rebar0.8L}} + 1.75 \cdot \frac{M_{LL0.8L} \cdot 12}{S_{rebar0.8L}}$$

$\boxed{f_1 = 15.99}$ ksi

$f_2 := 46.50$ ksi (Table E24-1.6-2)

$\boxed{\frac{f_1}{f_2} = 0.34}$

$$C_b := 1.75 - 1.05 \cdot \left(\frac{f_1}{f_2}\right) + 0.3 \cdot \left(\frac{f_1}{f_2}\right)^2 \quad < 2.3$$ $\boxed{C_b = 1.42}$

Therefore:

$$F_{nc} := C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$\boxed{F_{nc} = 58.72}$ ksi

$F_{nc} \le R_b \cdot R_h \cdot F_{yc}$ $\boxed{R_b \cdot R_h \cdot F_{yc} = 50.00}$ ksi

Use:

$F_{nc} := 50$ ksi

$\boxed{\phi_f \cdot F_{nc} = 50.00}$ ksi

$$M_{n\_1.0L} := F_{nc} \cdot S_{rebar} \cdot \left(\frac{1}{12}\right)$$ $M_{n\_1.0L} = 9883.01$ ft – kips

E45-4.13 Design Load Rating @ Pier

$$RF = \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC}\left(M_{DC\_neg}\right)}{\gamma_L\left(M_{LLIM\_neg}\right)}$$

Where:

Load Factors per Table 45.3-1

$\gamma_{Linv} := 1.75$

$\gamma_{Lop} := 1.35$

$\gamma_{DC} := 1.25$

Resistance Factors

$\phi := 1.0$ **MBE [6A.7.3]**

$\phi_c := 1.0$ per 45.3.7.4

$\phi_s := 1.0$ per 45.3.7.5

$M_{DC1\_neg} := M_{girder\_neg} + M_{deck\_neg} + M_{misc\_neg}$ $\quad M_{DC1\_neg} = -3073.22$ ft – kips

$M_{LLIM\_neg} := M_{LL\_neg}$ $\quad M_{LLIM\_neg} = -2414.17$ ft – kips

A. Strength Limit State

$$RF_{inv\_1.0L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot \left(-F_{nc}\right) - \gamma_{DC} \cdot \frac{M_{DC1\_neg} \cdot 12}{S_{NC\_neg}} - \gamma_{DC} \cdot \frac{M_{DC2\_neg} \cdot 12}{S_{rebar}}}{\gamma_{Linv} \cdot \left(\frac{M_{LLIM\_neg} \cdot 12}{S_{rebar}}\right)}$$

$RF_{inv\_1.0L} = 1.30$

$$RF_{op\_1.0L} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot \left(-F_{nc}\right) - \gamma_{DC} \cdot \frac{M_{DC1\_neg} \cdot 12}{S_{NC\_neg}} - \gamma_{DC} \cdot \frac{M_{DC2\_neg} \cdot 12}{S_{rebar}}}{\gamma_{Lop} \cdot \left(\frac{M_{LLIM\_neg} \cdot 12}{S_{rebar}}\right)}$$

$RF_{op\_1.0L} = 1.68$

B. Service II Limit State

$$RF = \frac{f_R - \gamma_D \cdot (f_D)}{\gamma_L \cdot (f_{LLIM})}$$

Allowable Flange Stress per **LRFD [6.10.4.2.2]**

$$f_R := 0.95 \cdot R_h \cdot F_y$$

$R_h := 1.0$ For non-hybrid sections

$f_R := 0.95 \cdot R_b \cdot R_h \cdot F_y$

$f_R = 47.50$ ksi

$f_D = f_{DC1} + f_{DC2}$

$$f_D := -\left[\left(\frac{M_{DC1\_neg} \cdot 12}{S_{NC\_neg}}\right) + \left(\frac{M_{DC2\_neg} \cdot 12}{S_{LT\_neg}}\right)\right]$$

$f_D = 17.68$ ksi

$$f_{LLIM} := \frac{-M_{LL\_neg} \cdot 12}{S_{rebar}}$$

$f_{LLIM} = 12.21$ ksi

Load Factors Per Table 45.3-1

$\gamma_D := 1.0$

$\gamma_{Lin} := 1.3$ Inventory

$\gamma_{Lop} := 1.0$ Operating

Inventory

$$RF_{inv\_1.0L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lin} \cdot f_{LLIM}}$$

$RF_{inv\_1.0L\_service} = 1.88$

Operating

$$RF_{op\_1.0L\_service} := \frac{f_R - \gamma_D \cdot f_D}{\gamma_{Lop} \cdot f_{LLIM}}$$

$RF_{op\_1.0L\_service} = 2.44$

E45-4.14 Rate for Shear - Negative Moment Region

Shear must be checked at each section of the girder. For this Rating example, shear is maximum at the pier, and will only be checked there for illustrative purposes.

The transverse intermediate stiffener spacing is 120". The spacing of the transverse intermediate stiffeners does not exceed 3D, therefore the section can be considered stiffened and the provisions of **LRFD [6.10.9.3]** apply.

$d_o := 120$ in

$D = 54.00$ in

$$k := 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \qquad k = 6.01$$

$$\frac{D}{t_w} = 108.00 \qquad \frac{D}{t_w} \geq 1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} \qquad 1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 82.67$$

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E_s \cdot k}{F_{yw}}\right) \qquad \boxed{C = 0.469}$$

The plastic shear force, V_p, is then:

$$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w \qquad \boxed{V_p = 783.00} \text{ kips}$$

$$V_n := V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}}\right] \qquad \boxed{V_n = 515.86} \text{ kips}$$

The factored shear resistance, V_r, is computed as follows **LRFD [6.10.9.1]**:

$\phi_v := 1.00$

$$V_r := \phi_v \cdot V_n \qquad \boxed{V_r = 515.86} \text{ kips}$$

HL-93 Maximum Shear @ Pier:

$$V_{DC1} := V_{girder} + V_{deck} + V_{misc} \qquad V_{DC1} = -108.84 \text{ kips}$$

$V_{DC2} = -12.03$ kips

$V_{LL} = -131.95$ kips

$M_{LLIM\_neg} = -2414.17$ ft – kips

E45-4.15 Design Load Rating @ Pier for Shear

$$RF = \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC}(V_{DC})}{\gamma_L(V_{LLIM})}$$

Where:

Load Factors per Table 45.3-1

$\gamma_{Linv} := 1.75$

$\gamma_{Lop} := 1.35$

$\gamma_{DC} := 1.25$

Resistance Factors

$\phi := 1.0$ **MBE [6A.7.3]**

$\phi_c := 1.0$ per 45.3.7.4

$\phi_s := 1.0$ per 45.3.7.5

A. Strength Limit State

Inventory

$$RF_{inv\_shear} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-V_n) - \gamma_{DC} \cdot (V_{DC1} + V_{DC2})}{\gamma_{Linv} \cdot (V_{LL})}$$

$$\boxed{RF_{inv\_shear} = 1.58}$$

Operating

$$RF_{op\_shear} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-V_n) - \gamma_{DC} \cdot (V_{DC1} + V_{DC2})}{\gamma_{Lop} \cdot (V_{LL})}$$

$$\boxed{RF_{op\_shear} = 2.05}$$

Since RF>1.0 @ operating for all checks, Legal Load Ratings are not required for this example.

E45-4.16 - Permit Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.12). Since the span lengths are less than 200', the lane loading requirements will not be considered for positive moments.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic load allowance is utilized. Future wearing surface shall not be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW. Future wearing surface shall be included in the check.

E45-4.16.1 - Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

The values from this analysis are used for performing the Wis-SPV design check per 45.12

Load Distribution Factors

Single Lane Interior DF - Moment $g_{m1} = 0.47$

Single Lane Interior DF - Shear $g_{v1} = 0.75$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.2$

$\gamma_{DC} := 1.25$ $\gamma_{DW} := 1.50$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

$$M_{0.4L} := \frac{g_{m1}}{1.2} \cdot 1.33 \cdot M_{pos} \qquad \boxed{M_{0.4L} = 1468.47} \text{ kip-ft}$$

$$M_{1.0L} := \left(\frac{g_{m1}}{1.2}\right) \cdot \left(\left(1.33 \cdot M_{neg}\right)\right) \qquad \boxed{M_{1.0L} = 1129.31} \text{ kip-ft}$$

$$V_{1.0L} := \left(\frac{g_{v1}}{1.2}\right)\cdot((1.33\cdot V_{max}))$$ $\boxed{V_{1.0L} = 128.28}$ kips

$$RF_{pos} := \frac{\phi\cdot\phi_c\cdot\phi_s\cdot M_{n\_0.4L} - \gamma_{DC}\cdot(M_{DC1} + M_{DC2}) - \gamma_{DW}\cdot M_{DW}}{\gamma_L\cdot(M_{0.4L})}$$

$\boxed{RF_{pos} = 2.55}$ $\boxed{RF_{pos}\cdot 190 = 483.65}$ kips

$$RF_{neg} := \frac{\phi\cdot\phi_c\cdot\phi_s\cdot M_{n\_1.0L} - \gamma_{DC}\cdot(-M_{DC1\_neg} - M_{DC2\_neg}) - \gamma_{DW}\cdot(-M_{DW\_neg})}{\gamma_L\cdot(M_{1.0L})}$$

$\boxed{RF_{neg} = 3.74}$ $\boxed{RF_{neg}\cdot 190 = 711.43}$ kips

$$RF_{shear} := \frac{\phi\cdot\phi_c\cdot\phi_s\cdot V_n - \gamma_{DC}\cdot[-(V_{DC1} + V_{DC2})] - \gamma_{DW}\cdot(-V_{DW})}{\gamma_L\cdot(V_{1.0L})}$$

$\boxed{RF_{shear} = 2.24}$ $\boxed{RF_{shear}\cdot 190 = 424.87}$ kips

424.87k > 190k minimum : CHECK OK

E45-4.16.2 - Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

For use with plans and rating sheet only.

Load Distribution Factors

Single Lane Interior DF - Moment $g_{m1} = 0.47$

Single Lane Interior DF - Shear $g_{v1} = 0.75$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.2$

$\gamma_{DC} := 1.25$ $\gamma_{DW} := 1.50$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance
or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

$M_{0.4L} := \frac{g_{m1}}{1.2} \cdot 1.33 \cdot M_{pos}$ $\boxed{M_{0.4L} = 1468.47}$ kip-ft

$M_{1.0L} := \left(\frac{g_{m1}}{1.2}\right) \cdot \left(\left(1.33 \cdot M_{neg}\right)\right)$ $\boxed{M_{1.0L} = 1129.31}$ kip-ft

$V_{1.0L} := \left(\frac{g_{v1}}{1.2}\right) \cdot \left(\left(1.33 \cdot V_{max}\right)\right)$ $V_{1.0L} = 128.28$ kips

$$RF_{pos1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot \left(M_{DC1} + M_{DC2}\right)}{\gamma_L \cdot \left(M_{0.4L}\right)}$$

$\boxed{RF_{pos1} = 2.68}$ $\boxed{RF_{pos1} \cdot 190 = 508.78}$ kips

$$RF_{neg1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC} \cdot \left(-M_{DC1\_neg} - M_{DC2\_neg}\right)}{\gamma_L \cdot \left(M_{1.0L}\right)}$$

$\boxed{RF_{neg1} = 4.16}$ $\boxed{RF_{neg1} \cdot 190 = 789.64}$ kips

$$RF_{shear1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC} \cdot \left[-\left(V_{DC1} + V_{DC2}\right)\right]}{\gamma_L \cdot \left(V_{1.0L}\right)}$$

$\boxed{RF_{shear1} = 2.37}$ $\boxed{RF_{shear1} \cdot 190 = 450.24}$ kips

E45-4.16.3 - Permit Rating with Multi-Lane Distribution w/o FWS

For use with plans and rating sheet only.

Load Distribution Factors

Multi Lane Interior DF - Moment $g_{m2} = 0.69$

Multi Lane Interior DF - Shear $g_{v2} = 0.93$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.3$

$\gamma_{DC} := 1.25$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance
or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

Multi Lane Ratings

$M_{0.4L} := g_{m2} \cdot 1.33 \cdot M_{pos}$ $\boxed{M_{0.4L} = 2600.09}$ kip-ft

$M_{1.0L} := g_{m2} \cdot (1.33 \cdot M_{neg})$ $\boxed{M_{1.0L} = 1999.56}$ kip-ft

$V_{1.0L} := g_{v2} \cdot (1.33 \cdot V_{max})$ $\boxed{V_{1.0L} = 191.88}$ kips

$$RF_{pos\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_L \cdot (M_{0.4L})}$$

$\boxed{RF_{pos\_ml} = 1.40}$ $\boxed{RF_{pos\_ml} \cdot 190 = 265.24}$ kips

$$RF_{neg\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC} \cdot \left(-M_{DC1\_neg} - M_{DC2\_neg}\right)}{\gamma_L \cdot \left(M_{1.0L}\right)}$$

$\boxed{RF_{neg\_ml} = 2.17}$ $\boxed{RF_{neg\_ml} \cdot 190 = 411.67}$ kips

$$RF_{shear\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC} \cdot \left[-\left(V_{DC1} + V_{DC2}\right)\right]}{\gamma_L \cdot \left(V_{1.0L}\right)}$$

$\boxed{RF_{shear\_ml} = 1.46}$ $\boxed{RF_{shear\_ml} \cdot 190 = 277.84}$ kips

E45-4.17 Summary of Rating

Steel Interior Girder							
Limit State		Design Load Rating		Legal Load Rating	Wis-SPV Ratings (kips)		
		Inventory	Operating		Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Strength I @ 0.4L	Flexure	1.41	1.82	N/A	484	509	265
	Shear	N/A	N/A	N/A	N/A	N/A	N/A
Strength I @ 1.0L	Flexure	1.30	1.68	N/A	711	790	412
	Shear	1.58	2.05	N/A	425	450	278
Service II	0.4L	1.38	1.80	N/A	Optional		Optional
	1.0L	1.88	2.44	N/A	Optional		Optional

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E45-5 Reinforced Concrete Slab Rating Example - LFR

Reference E45-1 for bridge data. For LFR, the Bureau of Structures rates concrete slab structures for the Design Load (HS20) and for Permit Vehicle Loads on an interior strip equal to one foot width.

This example calculates ratings of the controlling locations at the 0.4 tenths point of span 1 for positive moment and at the pier for negative moment.

E45-5.1 Design Criteria

Geometry:

$L_1 := 38.0$	ft	Span 1 Length
$L_2 := 51.0$	ft	Span 2 Length
$L_3 := 38.0$	ft	Span 3 Length
$slab_{width} := 42.5$	ft	out to out width of slab
$cover_{top} := 2.5$	in	concrete cover on top bars (includes 1/2in wearing surface)
$cover_{bot} := 1.5$	in	concrete cover on bottom bars
$d_{slab} := 17$	in	slab depth (not including 1/2in wearing surface)
$b := 12$	in	Interior strip width for analysis
$D_{haunch} := 28$	in	haunch depth (not including 1/2in wearing surface)
$A_{st\_0.4L} := 1.71$	$\frac{in^2}{ft}$	Area of longitudinal bottom steel at 0.4L (# 9's at 7in centers)
$A_{st\_pier} := 1.88$	$\frac{in^2}{ft}$	Area of longitudinal top steel at Pier (# 8's at 5in centers)

Material Properties:

$f'_c := 4$	ksi	concrete compressive strength
$f_y := 60$	ksi	yield strength of reinforcement

Weights:

$w_c := 150$	pcf	concrete unit weight
$w_{LF} := 387$	plf	weight of Type LF parapet (each)

E45-5.2 Analysis of an Interior Strip - one foot width

Use Strength Limit States to rate the concrete slab bridge. **MBE [6B.5.3.2]**

E45-5.2.1 Dead Loads

The slab dead load, D_{slab}, and the section properties of the slab, do not include the 1/2 inch wearing surface. But the 1/2 inch wearing surface load, D_{WS}, of 6 psf must be including in the analysis of the slab. For a one foot slab width:

$D_{WS} := 6$ 1/2 inch wearing surface load, plf

The parapet dead load is uniformly distributed over the full width of the slab when analyzing an Interior Strip. For a one foot slab width:

$$D_{para} := 2 \cdot \frac{w_{LF}}{slab_{width}} \qquad \boxed{D_{para} = 18} \quad \text{plf}$$

The unfactored dead load moments, M_D, due to slab dead load (D_{slab}), parapet dead load (D_{para}), and the 1/2 inch wearing surface (D_{ws}) are shown in Chapter 18 Example E18-1 (Table E18.4). For LFR, the total deal load moment (M_D) is the sum of the values M_{DC} and M_{DW} tabulated separately for LRFD calculations.

The structure was designed for a possible future wearing surface, D_{FWS}, of 20 psf.

$D_{FWS} := 20$ Possible wearing surface, plf

E45-5.2.2 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below.
The live loads to be placed on these widths are wheel loads (i.e., one line of wheels) or half of the lane load. The equivalent distribution width applies for both live load moment and shear.

Multi - Lane Loading:	$E = 48.0 + 0.06 \cdot S$	$\leq$ 84 in	**Std [3.24.3.2]**
Single - Lane Loading:	$E = 144$	in	**[45.6.2.1]**

where:

S = effective span length, in inches

For multi-lane loading:

(Span 1, 3) $E_{m13} := \min[84, 48.0 + 0.06\cdot(38\cdot12)]$

$\boxed{E_{m13} = 75.36}$ in

(Span 2) $E_{m2} := \min[84, 48.0 + 0.06\cdot(51\cdot12)]$

$\boxed{E_{m2} = 84.00}$ in

For single-lane loading:

(Span 1, 3) $\boxed{E_{s13} := 144.0}$ in

(Span 2) $\boxed{E_{s2} := 144.0}$ in

E45-5.2.3 Nominal Flexural Resistance: (M_n)

The depth of the compressive stress block, (a) is:

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$ **Std (8-17)**

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals:

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$ **Std (8-16)**

where:

d_s = slab depth (excl. 1/2" wearing surface) - bar clearance - 1/2 bar diameter

<u>Maximum Reinforcement Check</u>

The area of reinforcement to be used in calculating nominal resistance (M_n) shall not exceed 75 percent of the reinforcement required for the balanced conditions **MBE [6B.5.3.2]**.

$$\rho_b := 0.85^2 \cdot \left(\frac{f'_c}{f_y}\right) \cdot \frac{87}{87 + f_y} = 0.029$$ $$A_{smax} := \rho_b \cdot b \cdot d_s$$

E45-5.2.4 General Load - Rating Equation (for flexure)

$$RF = \frac{C - A_1 \cdot M_D}{A_2 \cdot M_L \cdot (1 + I)}$$ **MBE [6B.4.1]**

where:

$C := \phi \cdot M_n$

$\phi := 0.9$ **Std [8.16.1.2.2]**

A_1 = 1.3 for Dead Loads

A_2 = Live Load factor: 2.17 for Inventory, 1.3 for Operating

M_D = Unfactored Dead Load moments

M_L = Unfactored Live Load moments

I = Live Load Impact Factor (maximum 30%)

E45-5.2.5 Design Load (HS20) Rating

Equivalent Strip Width (E) and Distribution Factor (DF):

Use the multi-lane wheel distribution width for (HS20) live load.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E}$$ (where E is in feet)

Spans 1 & 3:

$$DF_{13} := \frac{12}{E_{m13}} = 0.159$$ wheels / ft-slab

Span 2:

$$DF_2 := \frac{12}{E_{m2}} = 0.143$$ wheels / ft-slab

Live Load Impact Factor (I)

$$I := \frac{50}{L + 125}$$ (maximum 0.3) **Std [3.8.2.1]**

Spans 1 & 3:

$$I_{13} := \min\left(0.3, \frac{50}{L_1 + 125}\right)$$ $\boxed{I_{13} = 0.3}$

Span 2:

$$I_2 := \min\left(0.3, \frac{50}{L_2 + 125}\right) \qquad \boxed{I_2 = 0.284}$$

Live Loads (LL)

The live loads shall be determined from live load analysis software using the higher of the HS20 Truck or Lane loads.

Rating for Flexure

$$RF = \frac{\phi \cdot M_n - 1.3 \cdot M_D}{A_2 \cdot M_L \cdot (1 + I)}$$

The Design Load Rating was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing limit state and location for the HS20 load is positive moment is in span 1 at the 0.4 pt.

Span 1 (0.4 pt.)

Flexural capacity:

$$A_{st\_0.4L} = 1.71 \quad \frac{in^2}{ft}$$

$$d_s := 17.0 - cover_{top} - \frac{9}{16} \qquad \boxed{d_s = 13.938} \quad in$$

$$a := \frac{A_{st\_0.4L} \cdot f_y}{0.85 \cdot f'_c \cdot b} \qquad \boxed{a = 2.51} \quad in$$

$$A_{smax} := \rho_b \cdot b \cdot d_s = 4.768 \qquad A_{smax} > A_{st\_0.4L} \quad OK$$

$$M_n := A_{st\_0.4L} \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \qquad \boxed{M_n = 1301} \quad kip\text{-}in$$

$$\boxed{M_n = 108.4} \quad kip\text{-}ft$$

The dead load consists of the slab self-weight and parapet weight divided evenly along the slab width:

$M_D := 18.1 \quad kip - ft$ (from Chapter 18 Example, Table E18.4)

The positive live load moment shall be the largest caused by the following (from live load analysis software):

Design Lane:	17.48 kip-ft
Design Truck:	24.01 kip-ft

Therefore:

$M_L := 24.01$ kip – ft

Inventory:

$$RF_i := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{2.17 \cdot M_L \cdot (1 + I_{13})} = 1.09$$

Inventory Rating = HS21

Operating:

$$RF_o := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{1.3 \cdot M_L \cdot (1 + I_{13})} = 1.82$$

Operating Rating = HS36

Rating for Shear:

Shear rating for concrete slab bridges may be ignored. Bending moment is assumed to control per **Std [3.24.4]**.

The Rating Factors, RF, for Inventory and Operating Rating are shown on the plans and also on the load rating summary sheet.

E45-5.2.6 Permit Vehicle Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per **[45.12]**.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution, and full dynamic load allowance is utilized. Future wearing surface will not be considered.

For a newly designed bridge, an additional check is required. The designer shall ensure that the results of a single-lane analysis utilizing the future searing surface are greater than 190 kips MVW.

E45-5.2.6.1 Wis-SPV Permit Rating with Multi Lane Distribution

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of the Pier.

The distribution width and impact factors are the same as calculated for the HS20 load.

At C/L of Pier

Flexural capacity:

$A_{st\_pier} = 1.88 \quad \frac{in^2}{ft}$

$d_{s\_pier} := 28.0 - cover_{top} - \frac{8}{16}$ $\boxed{d_{s\_pier} = 25}$ in

$a\_pier := \frac{A_{st\_pier} \cdot f_y}{0.85 \cdot f'_c \cdot b}$ $\boxed{a\_pier = 2.76}$ in

$A_{smax\_pier} := \rho_b \cdot b \cdot d_{s\_pier} = 8.552$ in^2 $A_{smax} > A_{st\_pier}$ OK

$M_{n\_pier} := A_{st\_pier} \cdot f_y \cdot \left(d_{s\_pier} - \frac{a\_pier}{2}\right)$ $\boxed{M_{n\_pier} = 2664.1}$ kip-in

$\boxed{M_{n\_pier} = 222}$ kip-ft

The dead load consists of the slab self-weight and parapet weight divided evenly along the slab width:

$M_{D\_pier} := 59.2$ kip-ft (from Chapter 18 Example, Table E18.4)

From live load analysis software, the live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing the maximum multi-lane distribution (at Spans 1 and 3) is:

$M_{LSPVm\_pier} := 66.06$ kip-ft

Annual Permit:

$$RF_{mpermit} := \frac{\phi \cdot M_{n\_pier} - 1.3 M_{D\_pier}}{1.3 \cdot M_{LSPVm\_pier} \cdot (1 + I_{13})} = 1.10$$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{mpermit} \cdot (190) = 209$ kips

E45-5.2.6.2 Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

The live load moment at the C/L of Pier due to the Wis-SPV with single-lane loading may be determined by scaling the live load moment from multi-lane loading:

$$M_{LSPVs\_pier} := M_{LSPVm\_pier} \cdot \frac{E_{m13}}{E_{s13}} = 34.57 \qquad \text{kip-ft}$$

Single-Trip Permit w/o FWS:

$$RF_{spermit} := \frac{\phi \cdot M_{n\_pier} - 1.3 M_{D\_pier}}{1.3 \cdot M_{LSPVs\_pier} \cdot (1 + I_{13})} = 2.10$$

The Wisconsin Standard Permit Vehicle (Wis-SPV) load that can be carried by the bridge is:

$$RF_{spermit} \cdot (190) = 399 \qquad \text{kips}$$

The Single-Lane MVW for the Wis-SPV is shown on the plans, up to a maximum of 250 kips. This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

E45-5.2.6.3 Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

From Chapter 18 Example, Table E18.4, the applied moment at the pier from the future wearing surface is:

$$M_{DW\_pier} := 4.9 \qquad \text{kip-ft}$$

Single-Trip Permit w/ FWS:

$$RF_{spermit\_fws} := \frac{\phi \cdot M_{n\_pier} - 1.3 \cdot (M_{D\_pier} + M_{DW\_pier})}{1.3 \cdot M_{LSPVs\_pier} \cdot (1 + I_{13})} = 1.99$$

The Wisconsin Standard Permit Vehicle (Wis-SPV) load that can be carried by the bridge is:

$$RF_{spermit\_fws} \cdot (190) = 379 \qquad \text{kips} \qquad > 190\text{k} \qquad \text{OK}$$

E45-5.3 Summary of Rating

Slab - Interior Strip					
Limit State	Design Load Rating		Permit Load Rating (kips)		
	Inventory	Operating	Multi DF w/o FWS	Single DF w/o FWS	Single DF w/ FWS
Flexure	HS21	HS36	209	399	379

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E45-6 Single Span PSG Bridge Rating Example - LFR

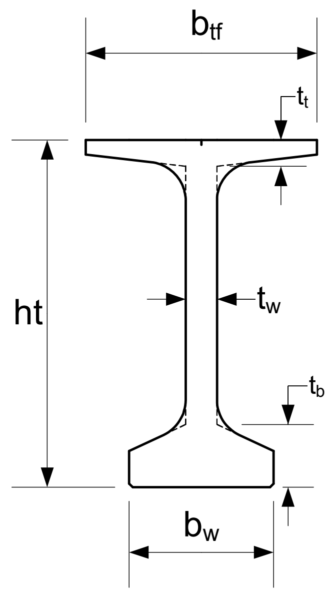
Reference E45-2 for bridge data. For LFR, the Bureau of Structures rates structures for the Design Load (HS20) and for Permit Vehicle loads. The rating below analyzes an interior girder only, which typically governs.

E45-6.1 Preliminary Data

$L := 146$	center to center of bearing, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{ci} := 6.8$	girder initial concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f'_s := 270$	strength of low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_s := 0.217$	area of strand, in2
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness (slab thickness - 1/2 in wearing surface), in
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$w_c := 0.150$	weight of concrete, kcf
$H_{avg} := 2$	average thickness of haunch, in
$S := 7.5$	spacing of the girders, ft
$ng := 6$	number of girders

E45-6.2 Girder Section Properties

72W Girder Properties (46 strands, 8 draped):



$b_{tf} := 48$	width of top flange, in
$t_t := 5.5$	avg. thickness of top flange, in
$t_w := 6.5$	thickness of web, in
$t_b := 13$	avg. thickness of bottom flange, in
$ht := 72$	height of girder, in
$b_w := 30$	width of bottom flange, in
$A_g := 915$	area of girder, in^2
$I_g := 656426$	moment of inertia of girder, in^4
$y_t := 37.13$	centroid to top fiber, in
$y_b := 34.87$	centroid to bottom fiber, in
$S_t := 17680$	section modulus for top, in^3
$S_b := 18825$	section modulus for bottom, in^3
$w_g := 0.953$	weight of girder, klf
$ns := 46$	number of strands
$e_s := 30.52$	centroid to cg strand pattern

$$e_g := y_t + 2 + \frac{t_{se}}{2} \qquad \boxed{e_g = 42.88} \text{ in}$$

Web Depth: $d_w := ht - t_t - t_b$ $\boxed{d_w = 53.50}$ in

$E_s := 28500$ Modulus of Elasticity of the Prestressing Strands, ksi

Concrete modulus of elasticity per WisDOT policy in **[19.3.3.8]**:

$E_{deck4} := 4125$ $\qquad E_D := E_{deck4}$

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351} \qquad E_B := E_{beam8}$$

$$E_{beam6.8} := 33000\,(.150)^{1.5} \cdot \sqrt{f'_{ci}} \qquad \boxed{E_{beam6.8} = 4999} \qquad E_{ct} := E_{beam6.8}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

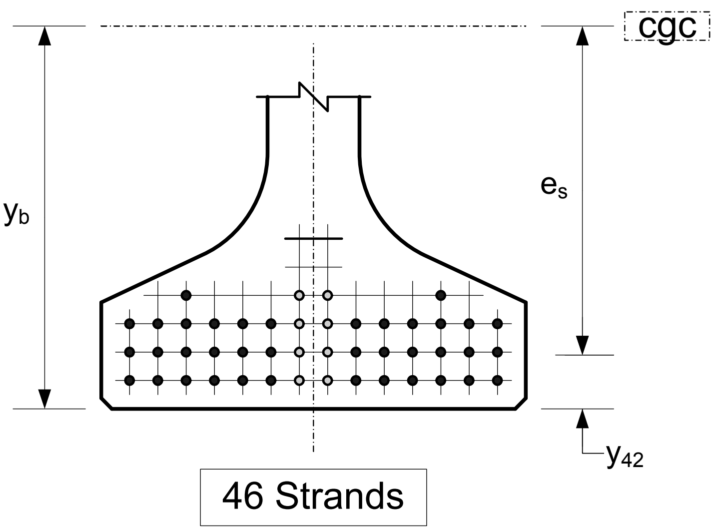


Figure E45-6.1

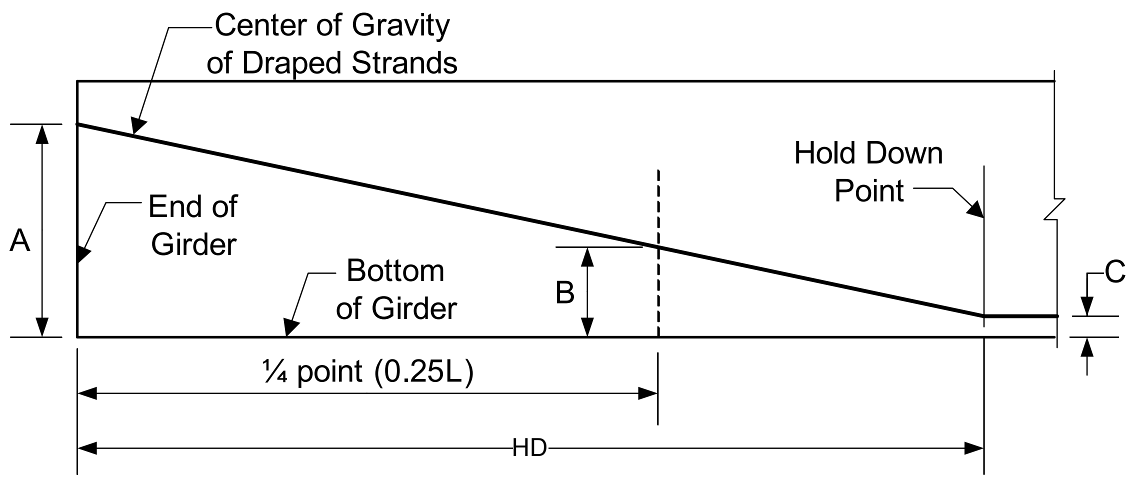


Figure E45-6.2

$A := 67$ in $\quad C := 5$ in $\quad B_{min} := 20.5$ in $\quad B_{max} := 23.5$ in

$$B_{avg} := \frac{B_{min} + B_{max}}{2} \qquad \boxed{B_{avg} = 22.0} \text{ in}$$

$$slope := \left[\frac{A - B_{avg}}{(0.25) \cdot L \cdot 12}\right] \cdot 100 \qquad \boxed{slope = 10.274} \ \%$$

$$HD := \frac{A - C}{\left(\frac{slope}{100}\right) \cdot 12} \qquad \boxed{HD = 50.29} \text{ ft}$$

E45-6.3 Composite Girder Section Properties

Calculate the effective flange width in accordance with **Std [9.8.3.1]**:

$$b_{eff} := \min\left[S \cdot 12, 12 \cdot t_{se} + t_w, \frac{(L \cdot 12)}{4}\right] \qquad \boxed{b_{eff} = 90} \quad \text{in}$$

The effective width, b_{eff}, must be adjusted by the modular ratio, n, to convert to the same concrete material (modulus) as the girder.

$$b_{eadj} := \frac{b_{eff}}{n} \qquad \boxed{b_{eadj} = 58.46} \quad \text{in}$$

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

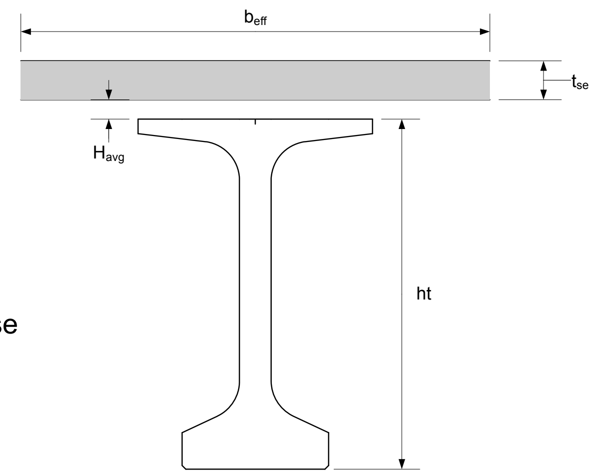
effective slab width; $\boxed{b_{eadj} = 58.46}$ in

haunch thickness; $\boxed{H_{avg} = 2.00}$ in

total height; $h_c := ht + H_{avg} + t_{se}$

$\boxed{h_c = 81.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	77.75	438	34089	2650458	2055	2652513
Girder	34.87	915	31906	1112564	656426	1768990
Haunch	73	0	0	0	0	0
Summation		1353	65996			4421503

$\Sigma A := 1353$ in^2

$\Sigma AY := 65996$ in^3

$\Sigma IplusAYsq := 4421503$ in^4

$$y_{cgb} := \frac{\Sigma AY}{\Sigma A}$$ $\boxed{y_{cgb} = 48.8}$ in

$$y_{cgt} := ht - y_{cgb}$$ $\boxed{y_{cgt} = 23.2}$ in

$$A_{cg} := \Sigma A$$ $\boxed{A_{cg} = 1353}$ in^2

$$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\,2}$$ $\boxed{I_{cg} = 1202381}$ in^4

$$S_{cgt} := \frac{I_{cg}}{y_{cgt}}$$ $\boxed{S_{cgt} = 51777}$ in^3

$$S_{cgb} := \frac{I_{cg}}{y_{cgb}}$$ $\boxed{S_{cgb} = 24650}$ in^3

E45-6.4 Dead Load Analysis - Interior Girder

Dead load on non-composite (D_1):

weight of 72W girders $\boxed{w_g = 0.953}$ klf

weight of 2-in haunch

$$w_h := \left(\frac{H_{avg}}{12}\right) \cdot \left(\frac{b_{tf}}{12}\right) \cdot (w_c)$$ $\boxed{w_h = 0.100}$ klf

weight of diaphragms $\boxed{w_D := 0.006}$ klf

weight of slab

$$w_d := \left(\frac{t_s}{12}\right) \cdot (S) \cdot (w_c)$$ $\boxed{w_d = 0.750}$ klf

$$D_1 := w_g + w_h + w_D + w_d$$ $\boxed{D_1 = 1.809}$ klf

$$V_{D1} := \frac{D_1 \cdot L}{2}$$ $\boxed{V_{D1} = 132.1}$ kips

$$M_{D1} := \frac{D_1 \cdot L^2}{8}$$ $\boxed{M_{D1} = 4820}$ kip-ft

* Dead load on composite (D_2):

weight of single parapet, klf $\boxed{w_p = 0.387}$ klf

weight of 2 parapets, divided equally to all girders, klf

$$D_2 := \frac{w_p \cdot 2}{ng} \qquad \boxed{D_2 = 0.129} \text{ klf}$$

$$V_{D2} := \frac{D_2 \cdot L}{2} \qquad \boxed{V_{D2} = 9.4} \text{ kips}$$

$$M_{D2} := \frac{D_2 \cdot L^2}{8} \qquad \boxed{M_{D2} = 344} \text{ kip-ft}$$

* Wearing Surface (DW): There is no current wearing surface on this bridge. However, it is designed for a 20 psf future wearing surface. Thus, it will be used in the calculations for the Wisconsin Standard Permit Vehicle Design Check, Section 45.12.

$$DW := \frac{w \cdot 0.020}{ng} \qquad \boxed{DW = 0.133} \text{ klf}$$

$$V_{DW} := \frac{DW \cdot L}{2} \qquad \boxed{V_{DW} = 9.7} \text{ kips}$$

$$M_{DW} := \frac{DW \cdot L^2}{8} \qquad \boxed{M_{DW} = 355} \text{ kip-ft}$$

* **Std [3.23.2.3.1.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

Total Unfactored Dead Load

$$V_D := V_{D1} + V_{D2} \qquad \boxed{V_D = 141.5} \text{ kips}$$

$$M_D := M_{D1} + M_{D2} \qquad \boxed{M_D = 5164} \text{ kip-ft}$$

E45-6.5 Live Load Analysis - Interior Girder

E45-6.5.1 Moment and Shear Distribution Factors for Interior Beams:

Moment and Shear Distribution Factors for interior girders are in accordance with **Std [3.23.1.2, 3.23.2.2]**:

For one Design Lane Loaded:

$$DF_s := \frac{S}{7}$$

$$\boxed{DF_s = 1.071}$$

For Two or More Design Lanes Loaded:

$$DF_m := \frac{S}{5.5}$$

$$\boxed{DF_m = 1.364}$$

E45-6.5.2 Live Load Moments

The live load load moments from analysis software (per wheel including impact with multi-lane distribution factor applied) are listed below:

Unfactored Live Load + Impact Moments per Wheel (kip-ft)		
Tenth Point	Truck	Lane
0	0	0
0.1	710	687
0.2	1250	1221
0.3	1620	1603
0.4	1839	1832
0.5	1896	1908

The HS20 lane load controls at midspan.

$M_{LLIM} := 1908$ kip-ft

E45-6.6 Determination of Pretress Losses

Calculate the components of the prestress losses; shrinkage, elastic shortening, creep and relaxation, using the approximate method in accordance with **Std [9.16.2]**.

Shrinkage

Relative Humidity $RH := 72$

$$SH := \frac{(17000 - 150 \cdot RH)}{1000}$$ $\boxed{SH = 6.200}$ ksi

Elastic Shortening

$E_{ci} := E_{beam6.8} = 4999$ $\boxed{E_{ci} = 4999}$ ksi

$A_{ps} := ns \cdot A_s = 9.982$ $\boxed{A_{ps} = 9.982}$ in^2

Estimated initial tendon stress:

$P_{si} := 0.69 \cdot A_{ps} \cdot f'_s = 1860$ $\boxed{P_{si} = 1860}$ kips

Dead load moment of girder:

$$M_g := 12 \cdot w_g \cdot \frac{L^2}{8} = 30471$$ $\boxed{M_g = 30471}$ k-in

According to **PCI Bridge Design Manual [18.5.4.3]**:

$$f_{cir} := \frac{P_{si}}{A_g} + \frac{\left(P_{si} \cdot e_s^{\,2}\right)}{I_g} - \frac{M_g \cdot e_s}{I_g}$$ $\boxed{f_{cir} = 3.255}$ ksi

$$ES := \frac{E_s}{E_{ci}} \cdot f_{cir}$$ $\boxed{ES = 18.553}$ ksi

Creep of Concrete

Moment due to concrete deck weight:

$$M_{slab} := 12 \cdot \frac{\left(w_d \cdot L^2\right)}{8} \qquad \boxed{M_{slab} = 23981} \quad \text{k-in}$$

Moment due to haunch weight:

$$M_{haunch} := 12 \cdot \frac{\left(w_h \cdot L^2\right)}{8} \qquad \boxed{M_{haunch} = 3197} \quad \text{k-in}$$

Moment due to diaphragms:

$$M_{nc} := 12 \cdot \frac{\left(w_D \cdot L^2\right)}{8} \qquad \boxed{M_{nc} = 191.8} \quad \text{k-in}$$

Moment due to composite DL:

$$M_c := M_{D2} \cdot 12 \qquad \boxed{M_c = 4125} \quad \text{k-in}$$

Centroid of composite section to C.G. of strand pattern:

$$e_c := e_s + \left(y_{cgb} - y_b\right) \qquad \boxed{e_c = 44.428} \quad \text{in}$$

Concrete stress at C.G. of strands due to all DL except girder:

$$f_{cds} := \left(M_{slab} + M_{haunch} + M_{nc}\right) \cdot \frac{e_s}{I_g} + M_c \cdot \frac{e_c}{I_{cg}} \qquad \boxed{f_{cds} = 1.425} \quad \text{ksi}$$

$$CR_c := 12 \cdot f_{cir} - 7 \cdot f_{cds} \qquad \boxed{CR_c = 29.080} \quad \text{ksi}$$

Relaxation of Prestressing Steel

$$CR_s := 5 - 0.10 \cdot ES - 0.05 \cdot \left(SH + CR_c\right) \qquad \boxed{CR_s = 1.381} \quad \text{ksi}$$

Total Prestress Losses

$$f_s := SH + ES + CR_c + CR_s \qquad \boxed{f_s = 55.214} \quad \text{ksi}$$

E45-6.7 Compute Nominal Flexural Resistance at Midspan

At failure, we can assume that the tendon stress is:

$$f_{su} := f'_s \cdot \left[1 - \left(\frac{\gamma}{\beta_1}\right) \cdot \left(\rho \cdot \frac{f'_s}{f'_{cd}}\right)\right]$$ **Std [9.17.4.1]**

where:

$\gamma := 0.28$	for low relaxation strands	**Std [9.1.2]**
$\beta_1 := 0.85$	for concrete deck in compression block, up to 4,000 psi	**Std [8.16.2.7]**

Calculation of ρ:

$A_{ps} = 9.982$ in^2

$b := b_{eff} = 90.000$ in

$d := y_t + H_{avg} + t_{se} + e_s$ $\boxed{d = 77.150}$ in

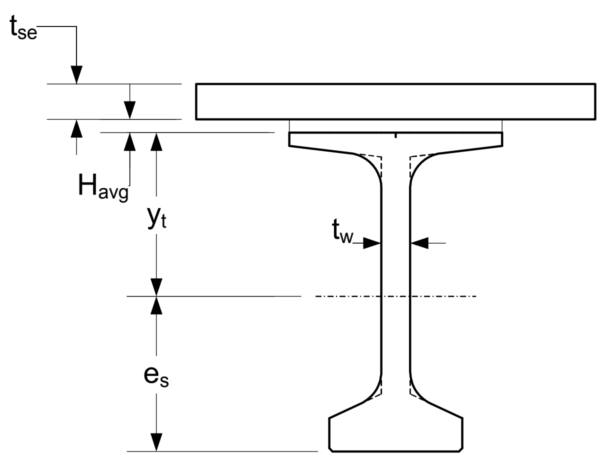


Figure E45-6.3

$$\rho := \frac{A_{ps}}{b \cdot d} = 0.00144$$

$\boxed{f_{su} = 261.4}$ ksi

Check the depth of the equivalent rectangular stress block, c, per **Std [9.17.2]**:

$$c := \frac{A_{ps} \cdot f_{su}}{0.85 f'_{cd} \cdot b}$$ $\boxed{c = 8.526}$ in

The calculated value of "c" is greater than the deck thickness, 7.5 in. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the capacity based upon a flanged section per **Std [9.17.3]**:

$$A_{sf} := 0.85 \cdot f'_{cd} \cdot \frac{(b - b_{tf}) \cdot t_{se}}{f_{su}}$$ $\boxed{A_{sf} = 4.098}$ in^2

$$A_{sr} := A_{ps} - A_{sf}$$ $\boxed{A_{sr} = 5.884}$ in^2

$$M_n := A_{sr} \cdot f_{su} \cdot d \cdot \left[1 - 0.6 \cdot \left(\frac{A_{sr} \cdot f_{su}}{b_{tf} \cdot d \cdot f'_{cd}}\right)\right] + 0.85 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot t_{se} \cdot (d - 0.5 \cdot t_{se})$$

$M_n = 189875$ k-in

$\boxed{M_n = 15823}$ k-ft

For prestressed concrete members, $\phi := 1.0$

$\boxed{\phi \cdot M_n = 15823}$ k-ft

Check Minimum Reinforcement

The amount of reinforcement must be sufficient to develop ϕM_n equal to 1.2 times the cracking moment M_{cr} per **Std [9.18.2.1]**. If $\phi M_n < 1.2 M_{cr}$, the nominal moment capacity shall be reduced according to **MBE [6B.5.3.3]**:

M_{cr} is calculated as follows:

$$M_{cr} := S_c \cdot (f_r + f_{pe}) - M_{dnc} \cdot \left[\left(\frac{S_c}{S_b}\right) - 1\right]$$

$$f_r := 7.5 \cdot \frac{\sqrt{f'_c \cdot 1000}}{1000}$$ **Std [9.15.2.3]** $\boxed{f_r = 0.671}$ ksi

$$M_{dnc} := 12 \cdot M_{D1}$$ $\boxed{M_{dnc} = 57841}$ kip-in

Effective prestress force after losses

$$P_{se} := A_{ps} \cdot (0.75 f'_s - f_s)$$ $\boxed{P_{se} = 1470}$ kips

$S_{nc} := S_b$ $\boxed{S_{nc} = 18825}$ in^3

$r := \sqrt{\frac{I_g}{A_g}}$ $\boxed{r = 26.784}$ in

$f_{pe} := \frac{P_{se}}{A_g} \cdot \left(1 + \frac{e_s \cdot y_b}{r^2}\right)$ $\boxed{f_{pe} = 3.990}$ ksi

$S_c := S_{cgb}$ $\boxed{S_c = 24650}$ in^3

$\boxed{1.2 \cdot M_{cr} = 9700}$ kip-ft $<$ $\phi \cdot M_n = 15823$ kip-ft

Therefore the requirement is satisfed.

E45-6.8 Compute Nominal Shear Resistance at First Critical Section

The following will illustrate the shear resistance calculation at the first critical section only. Due to the variation of resistances for shear along the length of the prestressed concrete I-beam, it is not certain what location will govern. Therefore, a systematic evaluation of the shear should be performed along the length of the beam.

The shear strength is the sum of contributions from nominal shear strength provided by concrete, V_c, and nominal shear strength provided by web reinforcement, V_s.

The critical section for shear is taken at a distance of H/2 from the face of the support per **Std [9.20.1.4]**.

$H := \frac{ht}{12} = 6.00$ ft $\boxed{\frac{H}{2} = 3.00}$ ft

The shear strength provided by concrete, V_c, is taken as the lesser of V_{ci} and V_{cw}:

$V_{ci} := 0.6 \cdot \sqrt{f'_c} \cdot b' \cdot d + V_d + \frac{V_i \cdot M_{cre}}{M_{max}} \geq 1.7 \sqrt{f'_c} \cdot b' \cdot d$ **Std [9.20.2.2]**

$f'_c = 8.000$ ksi

$b' := t_w = 6.500$ in

$V_d := (D_1 + D_2) \cdot \left(\frac{L}{2} - \frac{H}{2}\right) = 135.7$ k Shear due to unfactored dead load

$M_{cre} := \frac{I_{cg}}{Y_t} \cdot \left(6 \cdot \sqrt{f'_c} + f_{pe} - f_d\right)$ Moment causing flexural cracking at section due to externally applied loads

$M_{dnc} := \frac{\left(w_d + w_h + w_D + w_g\right) \cdot \left(\frac{H}{2}\right)}{2} \cdot \left(L - \frac{H}{2}\right) = 388.0$ k-ft Moment due to noncomposite dead load

$M_d := \frac{\left(D_1 + D_2\right) \cdot \left(\frac{H}{2}\right)}{2} \cdot \left(L - \frac{H}{2}\right) = 415.7$ k-ft Moment due to total unfactored dead load

$M_{dc} := \frac{\left(D_2\right) \cdot \left(\frac{H}{2}\right)}{2} \cdot \left(L - \frac{H}{2}\right) = 27.7$ k-ft Moment due to composite dead load

$f_d := \frac{M_{dnc} \cdot 12}{S_b} + \frac{M_{dc} \cdot 12}{S_{cgb}} = 0.261$ ksi Stress at extreme tension fiber due to unfactored dead load

Since there are draped strands for a distance of $HD = 50.289$ ft from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section.

$ns_{sb} := 38$ number of undraped strands

$ns_d := 8$ number of draped strands

Find the center of gravity for the 38 straight strands from the bottom of the girder:

$Y_{38S} := \frac{12 \cdot 2 + 12 \cdot 4 + 12 \cdot 6 + 2 \cdot 8}{ns_{sb}}$ $\boxed{Y_{38S} = 4.211}$ in

Find the center of gravity for the 8 draped strands from the bottom of the girder:

$slope = 10.274$ %

$Y_{8D} := A - \frac{H}{2} \cdot 12 \cdot \left(\frac{slope}{100}\right)$ $\boxed{Y_{8D} = 63.301}$ in

Find the combined center of gravity for all strands from the bottom of the girder:

$Y_{COMB} := \frac{ns_{sb} \cdot Y_{38S} + ns_d \cdot Y_{8D}}{ns_{sb} + ns_d}$ $\boxed{Y_{COMB} = 14.487}$ in

Find the distance from the girder's centroid to the center of gravity of strands:

$e_{s\_crit} := y_b - Y_{COMB}$ $\boxed{e_{s\_crit} = 20.38}$ in

The shear depth from top of composite section to center of gravity of strands:

$d_v := \max\left(0.8 \cdot H, y_t + H_{avg} + t_{se} + e_{s\_crit}\right)$ $\boxed{d_v = 67.0}$ in

Find the revised value of fpe at the critical shear location:

$$f_{pe} := \frac{P_{se}}{A_g} \cdot \left(1 + \frac{e_{s\_crit} \cdot y_b}{r^2}\right)$$ $\boxed{f_{pe} = 3.199}$ ksi

Therefore:

$Y_t := y_{cgb} = 48.778$ in

$$M_{cre} := \frac{I_{cg}}{Y_t} \cdot \left(6 \cdot \frac{\sqrt{f'_c \cdot 1000}}{1000} + f_{pe} - f_d\right) \cdot \left(\frac{1}{12}\right)$$ $\boxed{M_{cre} = 7137}$ k-ft

From live load analysis software:

$M_l := 159.71$ k-ft — from HS20 lane load at crit. section

$M_u := 1.3M_d + 2.17 \cdot M_l = 887.0$ k-ft — Maximum factored moment at section

$M_{max} := M_u - M_d = 471.3$ k-ft — Maximum factored moment due to externally applied loads

$V_{u\_sim} := 291.6$ k — Maximum factored shear occurring simultaneously with M_{max}

$V_i := V_{u\_sim} - V_d$ $\boxed{V_i = 155.9}$ kips

Therefore:

$$V_{ci} := \max\left(0.6 \cdot \frac{\sqrt{f'_c \cdot 1000}}{1000} \cdot b' \cdot d_v + V_d + \frac{V_i \cdot M_{cre}}{M_{max}}, 1.7\sqrt{f'_c} \cdot b' \cdot d_v\right)$$

$\boxed{V_{ci} = 2520.7}$ kips

$$V_{cw} := \left(3.5\sqrt{f'_c} + 0.3 \cdot f_{pc}\right) \cdot b' \cdot d_v + V_p$$

$$f_{pc} := \frac{P_{se}}{A_g} - \frac{P_{se} \cdot e_{s\_crit} \cdot \left(y_{cgb} - y_b\right)}{I_g} + \frac{12M_{dnc} \cdot \left(y_{cgb} - y_b\right)}{I_g}$$

$\boxed{f_{pc} = 1.071}$ ksi

$$V_p := \frac{ns_d}{ns} \cdot P_{se} \cdot \frac{slope}{100} = 26.269$$ $\boxed{V_p = 26.3}$ kips

$\boxed{V_{cw} = 302.5}$ kips

$V_c := \min\left(V_{ci}, V_{cw}\right)$ $\boxed{V_c = 302.5}$ kips

Shear strength provided by web reinforcement:

Calculate the shear resistance at H/2:

$s := 18$ in

$A_v := 0.40$ in^2 for #4 rebar stirrups

A more refined analysis using average spacing across multiple stirrup zones may be used (refer to **MBE [6A.5.8, 2015 Interim Revisions]**, however this example conservatively considers the maximum spacing between the current and adjacent analysis points.

$f_y := 60$ ksi

$d_v = 67.01$ in

$$V_s := \min\left(A_v \cdot f_y \cdot \frac{d_v}{s}, 8 \cdot \frac{\sqrt{f'_c \cdot 1000}}{1000} \cdot b' \cdot d_v\right) \qquad \boxed{V_s = 89.4} \text{ kips}$$

The nominal shear capacity is:

$\phi_v := 0.9$

$V_n := V_c + V_s = 391.9$ kips

$\boxed{\phi_v \cdot V_n = 352.7}$ kips

E45-6.9 Design Load Rating

The inventory rating checks include Concrete Tension, Concrete Compression, Prestressing Steel Tension, and Flexural and Shear Strength. The operating rating checks include Prestressing Steel Tension and Flexural and Shear Strength. Refer to per **MBE [6B.5.3.3].**

Unfactored stress due to prestress force after losses:

$$F_{p\_bot} := \frac{-P_{se}}{A_g} \cdot \left(1 + \frac{e_s \cdot y_b}{r^2}\right) \qquad \boxed{F_{p\_bot} = -3.990} \text{ ksi}$$

$$F_{p\_top} := \frac{-P_{se}}{A_g} \cdot \left(1 - \frac{e_s \cdot y_t}{r^2}\right) \qquad \boxed{F_{p\_top} = 0.931} \text{ ksi}$$

Unfactored dead load stress:

$$F_{d\_bot} := \frac{12 \cdot M_{D1}}{S_b} + \frac{12 \cdot M_{D2}}{S_{cgb}} \qquad \boxed{F_{d\_bot} = 3.240} \quad \text{ksi}$$

$$F_{d\_top} := \frac{-12 \cdot M_{D1}}{S_t} - \frac{12 M_{D2}}{S_{cgt}} \qquad \boxed{F_{d\_top} = -3.351} \quad \text{ksi}$$

Secondary prestress forces (assumed):

$$F_s := 0$$

Unfactored live load stress including impact:

$$F_{L\_bot} := \frac{12 M_{LLIM}}{S_{cgb}} \qquad \boxed{F_{L\_bot} = 0.929} \quad \text{ksi}$$

$$F_{L\_top} := \frac{-12 M_{LLIM}}{S_{cgt}} \qquad \boxed{F_{L\_top} = -0.442} \quad \text{ksi}$$

Concrete Tension Rating:

$$RF_{inv\_t} := \frac{6 \frac{\sqrt{f'_c \cdot 1000}}{1000} - \left(F_{d\_bot} + F_{p\_bot} + F_s\right)}{F_{L\_bot}} \qquad \boxed{RF_{inv\_t} = 1.386}$$

Concrete Compression Rating:

$$RF_{inv\_c1} := \frac{-0.6 \cdot f'_c - \left(F_{d\_top} + F_{p\_top} + F_s\right)}{F_{L\_top}} \qquad \boxed{RF_{inv\_c1} = 5.382}$$

$$RF_{inv\_c2} := \frac{-0.4 \cdot f'_c - 0.5 \cdot \left(F_{d\_top} + F_{p\_top} + F_s\right)}{F_{L\_top}} \qquad \boxed{RF_{inv\_c2} = 4.500}$$

Prestressing Steel Tension Rating:

$$f'_y := 0.9 \cdot f'_s \qquad \boxed{f'_y = 243.0} \quad \text{ksi}$$

$$N := \text{round}\left(\frac{E_s}{E_{beam8}}\right) \qquad \boxed{N = 4}$$

$$F_{d\_ps} := N \cdot \left(M_g + M_{slab} + M_{haunch} + M_{nc}\right) \cdot \frac{e_s}{I_g} + N \cdot M_c \cdot \frac{e_c}{I_{cg}}$$

$\boxed{F_{d\_ps} = 11.367}$ ksi

$$F_{p\_ps} := \frac{P_{se}}{A_{ps}}$$ $\boxed{F_{p\_ps} = 147.286}$ ksi

$$F_{L\_ps} := N \cdot 12 \cdot M_{LLIM} \cdot \frac{e_c}{I_{cg}}$$ $\boxed{F_{L\_ps} = 3.384}$ ksi

$$RF_{inv\_ps\_tens} := \frac{0.8 \cdot f'_y - \left(F_{d\_ps} + F_{p\_ps} + F_s\right)}{F_{L\_ps}}$$ $\boxed{RF_{inv\_ps\_tens} = 10.564}$

$$RF_{op\_ps\_tens} := \frac{0.9 \cdot f'_y - \left(F_{d\_ps} + F_{p\_ps} + F_s\right)}{F_{L\_ps}}$$ $\boxed{RF_{op\_ps\_tens} = 17.744}$

Flexural Strength Rating:

$$RF_{inv\_m} := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{2.17 \cdot M_{LLIM}}$$ $\boxed{RF_{inv\_m} = 2.200}$

$$RF_{op\_m} := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{1.3 \cdot M_{LLIM}}$$ $\boxed{RF_{op\_m} = 3.673}$

Shear Strength Rating:

$V_L := 56.86$ kips from LL analysis software

$$RF_{inv\_v} := \frac{\phi_v \cdot V_n - 1.3 \cdot V_d}{2.17 \cdot V_L}$$ $\boxed{RF_{inv\_v} = 1.429}$

$$RF_{op\_v} := \frac{\phi_v \cdot V_n - 1.3 \cdot V_d}{1.3 \cdot V_L}$$ $\boxed{RF_{op\_v} = 2.385}$

E45-6.10 Permit Load Rating

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per 45.12.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming that full dynamic allowance is utilized. Future wearing surface shall be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW.

From live load analysis software, the force effects with distribution factor and impact included are:

$M190_{LLm} := 3985.01$ $\quad M190_{LLs} := 3131.08$ $\quad$ kip-ft per girder at midspan

$V190_{LLm} := 120.55$ $\quad V190_{LLs} := 94.72$ $\quad$ kips at $\frac{H}{2} = 3$ ft

$$F_{L\_ps\_190m} := N \cdot 12 \cdot M190_{LLm} \cdot \frac{e_c}{I_{cg}} \qquad \boxed{F_{L\_ps\_190m} = 7.068}$$

$$F_{L\_ps\_190s} := N \cdot 12 \cdot M190_{LLs} \cdot \frac{e_c}{I_{cg}} \qquad \boxed{F_{L\_ps\_190s} = 5.553}$$

Additional dead load from wearing surface at midspan:

$\boxed{M_{DW} = 355.3}$ kip-ft

Additional dead load from wearing surface at critical shear section:

$$V_{DW} := DW \cdot \left(\frac{L}{2} - \frac{H}{2}\right) \qquad \boxed{V_{DW} = 9.33} \text{ kips}$$

$$F_{dw\_ps} := N \cdot \left(12 M_{DW}\right) \cdot \frac{e_c}{I_{cg}} \qquad \boxed{F_{dw\_ps} = 0.630} \text{ ksi}$$

Multi-Lane w/o Future Wearing Surface:

$$RF_{190m\_ps\_t} := \frac{0.9 \cdot f'_y - (F_{d\_ps} + F_{p\_ps} + F_s)}{F_{L\_ps\_190m}}$$ $\boxed{RF_{190m\_ps\_t} = 8.496}$

$$RF_{op\_190m\_m} := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{1.3 \cdot M190_{LLm}}$$ $\boxed{RF_{op\_190m\_m} = 1.759}$

$$RF_{op\_190m\_v} := \frac{\phi_v \cdot V_n - 1.3 \cdot V_d}{1.3 \cdot V190_{LLm}}$$ $\boxed{RF_{op\_190m\_v} = 1.125}$

Single-Lane w/o Future Wearing Surface:

$$RF_{190s\_ps\_t} := \frac{0.9 \cdot f'_y - (F_{d\_ps} + F_{p\_ps} + F_s)}{F_{L\_ps\_190s}}$$ $\boxed{RF_{190s\_ps\_t} = 10.813}$

$$RF_{op\_190s\_m} := \frac{\phi \cdot M_n - 1.3 \cdot M_D}{1.3 \cdot M190_{LLs}}$$ $\boxed{RF_{op\_190s\_m} = 2.238}$

$$RF_{op\_190s\_v} := \frac{\phi_v \cdot V_n - 1.3 \cdot V_d}{1.3 \cdot V190_{LLs}}$$ $\boxed{RF_{op\_190s\_v} = 1.432}$

Single-Lane w/ Future Wearing Surface:

$$RF_{190sws\_ps\_t} := \frac{0.9 \cdot f'_y - (F_{d\_ps} + F_{dw\_ps} + F_{p\_ps} + F_s)}{F_{L\_ps\_190s}}$$

$\boxed{RF_{190sws\_ps\_t} = 10.700}$

$$RF_{op\_190sws\_m} := \frac{\phi \cdot M_n - 1.3 \cdot (M_D + M_{DW})}{1.3 \cdot M190_{LLs}}$$ $\boxed{RF_{op\_190sws\_m} = 2.125}$

$$RF_{op\_190sws\_v} := \frac{\phi_v \cdot V_n - 1.3 \cdot (V_d + V_{DW})}{1.3 \cdot V190_{LLs}}$$ $\boxed{RF_{op\_190sws\_v} = 1.333}$

E45-6.11 Summary of Rating Factors

Interior Girder							
Limit State		Design Load Rating		Permit Load Rating (kips)		
		Inventory	Operating	Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Strength	Flexure	HS 44	HS 73	403	425	334
	Shear	HS 28	HS 47	253	272	213
Service	Concrete Tension	HS 27	N/A	N/A	N/A	N/A
	Concrete Compression 1	HS 107	N/A	N/A	N/A	N/A
	Concrete Compression 2	HS 90	N/A	N/A	N/A	N/A
	Steel Tension	HS 211	HS 354	2033	2068	1614

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E45-7 Two Span 54W" Prestressed Girder Bridge - Continuity Reinforcement, Rating Example - LFR

Reference E45-3 for bridge data. For LFR, the Bureau of Structures rates structures for the Design Load (HS20) and for Permit Vehicle loads. The rating below analyzes an interior girder only, which typically governs. The rating below analyzes an interior girder only in the negative moment region (continuity reinforcement).

E45-7.1 Design Criteria

$L := 130$	center of bearing at abutment to CL pier for each span, ft
$L_g := 130.375$	total length of the girder (the girder extends 6 inches past the center of bearing at the abutment and 1.5" short of the center line of the pier).
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_y := 60$	yield strenght of mild reinforcement, ksi
$E_s := 29000$	ksi, Modulus of Elasticity of the reinforcing steel
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$w_c := 0.150$	kcf
$h := 2$	height of haunch, inches

E45-7.2 Modulus of Elasticity of Beam and Deck Material

The modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad E_B := E_{beam8} \qquad \boxed{E_B = 6351}$$

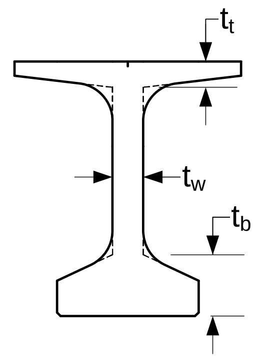
$$E_D := E_{deck4} \qquad \boxed{E_D = 4125}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

E45-7.3 Section Properties

54W Girder Properties:

$w_{tf} := 48$	in
$t_w := 6.5$	in
$ht := 54$	in
$b_w := 30$	width of bottom flange, in
$A_g := 798$	in^2
$I_g := 321049$	in^4
$y_t := 27.70$	in
$y_b := -26.30$	in



E45-7.4 Girder Layout

$S := 7.5$	Girder Spacing, feet
$ng := 6$	Number of girders

E45-7.5 Loads

$w_g := 0.831$	weight of 54W girders, klf
$w_d := 0.100$	weight of 8-inch deck slab (interior), ksf
$w_h := 0.100$	weight of 2-in haunch, klf
$w_{di} := 0.410$	weight of each diaphragm on interior girder (assume 2), kips
$w_{ws} := 0.020$	future wearing surface, ksf
$w_p = 0.387$	weight of parapet, klf

E45-7.5.1 Dead Loads

Dead load on non-composite (D_1):

interior:

$$w_{D1} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L}$$ $\boxed{w_{D1} = 1.687}$ klf

* Dead load on composite (D_2):

$$w_{D2} := \frac{2 \cdot w_p}{ng}$$ $\boxed{w_{D2} = 0.129}$ klf

* Wearing Surface (DW):

$$w_{DW} := \frac{w \cdot w_{ws}}{ng}$$ $\boxed{w_{DW} = 0.133}$ klf

* **Std [3.23.2.3.1.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E45-7.5.2 Live Load Analysis

Load Distribution to Interior Girders

Moment and Shear Distribution Factors for interior girders are in accordance with **Std [3.23.1.2, 3.23.2.2]**:

For one Design Lane Loaded:

$$DF_s := \frac{S}{7}$$ $\boxed{DF_s = 1.071}$

For Two or More Design Lanes Loaded:

$$DF_m := \frac{S}{5.5}$$ $\boxed{DF_m = 1.364}$

E45-7.6 Dead Load Moments

The unfactored dead load moments are listed below (values are in kip-ft):

Unfactored Dead Load Interior Girder Moments, (ft-kips)			
Tenth Point	D1 non-composite	D2 composite	DW composite
0.5	3548	137	141
0.6	3402	99	102
0.7	2970	39	40
0.8	2254	-43	-45
0.9	1253	-147	-151
1.0	0	-272	-281

The D_1 values are the component non-composite dead loads and include the weight of the girder, haunch, diaphragms and the deck.

The D_2 values are the component composite dead loads and include the weight of the parapets.

The DW values are the composite dead loads from the future wearing surface.

Note that the girder dead load moments (a portion of D_1) are calculated based on the CL bearing to CL bearing length. The other D_1 moments are calculated based on the span length (center of bearing at the abutment to centerline of the pier).

The total combined dead load is equal to:

$M_{DL} := -\left(M_{D1} + M_{D2}\right)$ $\boxed{M_{DL} = 272.0}$ kips without wearing surface

$M_{DL\_WS} := -\left(M_{D1} + M_{D2} + M_{DW}\right)$ $\boxed{M_{DL\_WS} = 553.0}$ kips with wearing surface

E45-7.7 Live Load Moments

The unfactored live load load moments (including distribution factor and impact) are listed below (values are in kip-ft) for the HS20 truck and lane loads.

Unfactored Live Load + Impact Moments per Lane (kip-ft)		
Tenth Point	HS20 Truck	HS20 Lane
0.5	-358	-365
0.6	-430	-438
0.7	-501	-511
0.8	-573	-584
0.9	-644	-875
1	-716	-1459

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$M_{LL} := 1459$ kip-ft

E45-7.8 Composite Girder Section Properties

Calculate the effective flange width in accordance with Chapter 17.2.11.

The effective flange width in accordance with **Std [9.8.3.1]**:

$$w_e := \min\left[S \cdot 12, 12 \cdot t_{se} + t_w, \frac{(L \cdot 12)}{4}\right] \qquad \boxed{w_e = 90.00} \text{ in}$$

The effective width, w_e, must be adjusted by the modular ratio, $n = 1.54$, to convert to the same concrete material (modulus) as the girder.

$$w_{eadj} := \frac{w_e}{n} \qquad \boxed{w_{eadj} = 58.46} \text{ in}$$

Calculate the composite girder section properties:

effective slab thickness; $t_{se} = 7.50$ in

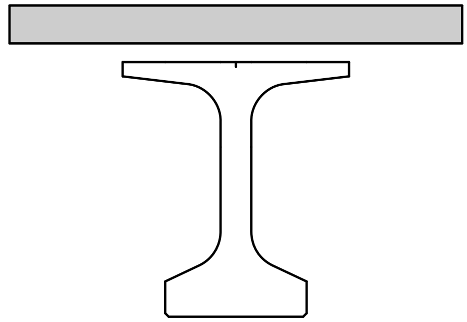
effective slab width; $W_{eadj} = 58.46$ in

haunch thickness; $h = 2.0$ in

total height; $h_c := ht + h + t_{se}$

$h_c = 63.50$ in

$n = 1.540$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	$I+AY^2$
Deck	59.75	438	26197	1565294	2055	1567349
Girder	26.3	798	20987	551969	321049	873018
Haunch	55	0	0	0	0	0
Summation		1236	47185			2440367

$\Sigma A := 1236$ in^2

$\Sigma AY := 47185$ in^4

$\Sigma IplusAYsq := 2440367$ in^4

$$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$$ $y_{cgb} = -38.2$ in

$y_{cgt} := ht + y_{cgb}$ $y_{cgt} = 15.8$ in

$A_{cg} := \Sigma A$ in^2

$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^2$ $I_{cg} = 639053$ in^4

Deck:

$$S_c := n \cdot \frac{I_{cg}}{y_{cgt} + h + t_{se}}$$ $S_c = 38851$ in^4

E45-7.9 Flexural Strength Capacity at Pier

All of the continuity reinforcement is placed in the top mat. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

$bar_{trans} := 5$ (transverse bar size)

$Bar_D(bar_{trans}) = 0.625$ in (transverse bar diameter)

$Bar_{No} = 10$

$Bar_D(Bar_{No}) = 1.27$ in (Assumed bar size)

$$d_e := ht + h + t_s - cover - Bar_D(bar_{trans}) - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_e = 60.24} \text{ in}$$

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

The continuity reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches.

From E19-2, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement is placed at 1/2 of this bar spacing, .

#10 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 3.57}$ in²/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12} \qquad \boxed{As = 26.80} \text{ in}^2$$

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

$$a := \frac{As \cdot f_y}{0.85 \cdot b_w \cdot f'_c} \qquad \boxed{a = 7.883} \text{ in}$$

This is approximately equal to the thickness of the bottom flange height of 7.5 inches. Therefore rectangular section strength calculation may be used.

$$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12} \qquad \boxed{M_n = 7544} \text{ kip-ft}$$

$$\boxed{\phi_f \cdot M_n = 6790} \text{ kip-ft}$$

E45-7.10 Design Load Rating

This design example illustrates the rating checks required at the location of maximum negative moment. These checks are also required at the locations of continuity bar cut offs but are not shown here.

$$RF_{inv} := \frac{\phi_f \cdot M_n - 1.3 \cdot M_{DL}}{2.17 \cdot M_{LL}} \qquad \boxed{RF_{inv} = 2.033}$$

$$RF_{op} := \frac{\phi_f \cdot M_n - 1.3 \cdot M_{DL}}{1.3 \cdot M_{LL}} \qquad \boxed{RF_{op} = 3.393}$$

E45-7.11 Permit Load Rating

Check the Wisconsin Standard Permit Vehicle per 45.12

For a symmetric 130' two span structure:

$MSPV_{LL} := 1029.8$ kip-ft per wheel line without impact

Per **Std [3.8.2.1]**:

$$IMPACT := \min\left(0.3, \frac{50}{L + 125}\right) \qquad \boxed{IMPACT = 0.196}$$

Single Lane Distribution per Girder with Impact:

$$MSPV_{LLIMs} := MSPV_{LL} \cdot DF_s \cdot (1 + IMPACT) \qquad \boxed{MSPV_{LLIMs} = 1319.7} \text{ kip-ft}$$

Multi Lane Distribution per Girder with Impact:

$$MSPV_{LLIMm} := MSPV_{LL} \cdot DF_m \cdot (1 + IMPACT) \qquad \boxed{MSPV_{LLIMm} = 1679.6} \text{ kip-ft}$$

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per 45.12.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming that full dynamic allowance is utilized. Future wearing surface shall be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW.

Single Lane Distribution w/ FWS

$$RF_{SPVsws} := \frac{\phi_f \cdot M_n - 1.3 \cdot M_{DL\_WS}}{1.3 \cdot MSPV_{LLIMs}}$$ $\boxed{RF_{SPVsws} = 3.539}$

$$Wt_{SPVsws} := RF_{SPVsws} \cdot 190$$ $\boxed{Wt_{SPVsws} = 672.4}$ kips >> 190 kips, OK

Single Lane Distribution w/o FWS

The rating for the Wis-SPV vehicle is now checked without the Future Wearing Surface. This value is reported on the plans.

$$RF_{SPVs} := \frac{\phi_f \cdot M_n - 1.3 \cdot M_{DL}}{1.3 \cdot MSPV_{LLIMs}}$$ $\boxed{RF_{SPVs} = 3.752}$

$$Wt_{SPVs} := RF_{SPVs} \cdot 190$$ $\boxed{Wt_{SPVs} = 712.8}$ kips

Since this value is greater than 250 kips, 250 kips is reported on the plans and on the Bridge Load Rating Summary form for the single-lane Permit Load Rating.

Multi-Lane Distribution w/o FWS

$$RF_{SPVm} := \frac{\phi_f \cdot M_n - 1.3 \cdot M_{DL}}{1.3 \cdot MSPV_{LLIMm}}$$ $\boxed{RF_{SPVm} = 2.948}$

$$Wt_{SPVm} := RF_{SPVm} \cdot 190$$ $\boxed{Wt_{SPVm} = 560.1}$ kips

Since this value is greater than 250 kips, 250 kips is reported on the Bridge Load Rating Summary form for the multi-lane Permit Load Rating.

E45-7.12 Summary of Rating Factors

Interior Girder						
Limit State		Design Load Rating		Legal Load	Permit Load Rating (kips)	
		Inventory	Operating	Rating	Single Lane	Multi-Lane
Strength 1	Flexure	HS 40	HS 67	N/A	250	250

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E45-8 Steel Girder Rating Example - LFR

Reference E45-4 for bridge data. For LFR, the Bureau of Structures rates structures for the Design Load (HS20) and for Permit Vehicle loads. The rating below analyzes an interior girder only, which typically governs.

E45-8.1 Preliminary Data

$N_{spans} := 2$		Number of spans
$L := 120$	ft	span length
$N_b := 5$		number of girders
$S := 9.75$	ft	girder spacing
$L_b := 240$	in	cross-frame spacing
$F_{yw} := 50$	ksi	web yield strength
$F_{yf} := 50$	ksi	flange yield strength
$f'_c := 4.0$	ksi	concrete 28-day compressive strength
$f_y := 60$	ksi	reinforcement strength
$E_s := 29000$	ksi	modulus of elasticity
$t_{deck} := 9.0$	in	total deck thickness
$t_s := 8.5$	in	effective deck thickness when 1/2" wearing surface is removed from total deck thickness
$w_s := 0.490$	kcf	steel density **Std [3.3.6]**
$w_c := 0.150$	kcf	concrete density **Std [3.3.6]**
$w_{misc} := 0.030$	kip/ft	additional miscellaneous dead load (per girder) per 17.2.4.1
$w_{par} := 0.387$	kip/ft	parapet weight (each)
$w_{deck} := 46.5$	ft	deck width
$d_{haunch} := 3.5$	in	haunch depth (from top of web for design) (for construction, the haunch is measured from the top of the top flange)

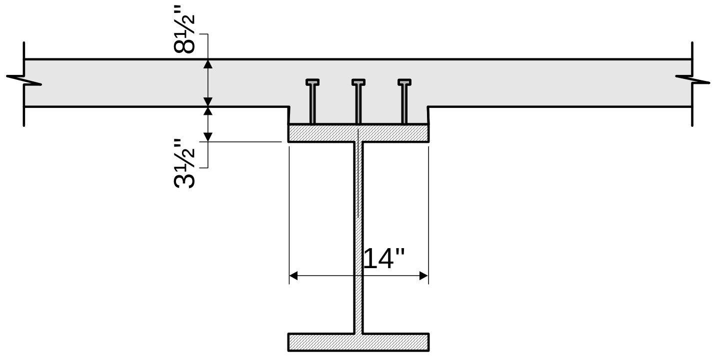

Figure E45-8.1-1
Composite Cross Section at Location of Maximum Positive Moment (0.4L)
(Note: 1/2" Intergral Wearing Surface has been removed for structural calcs.)

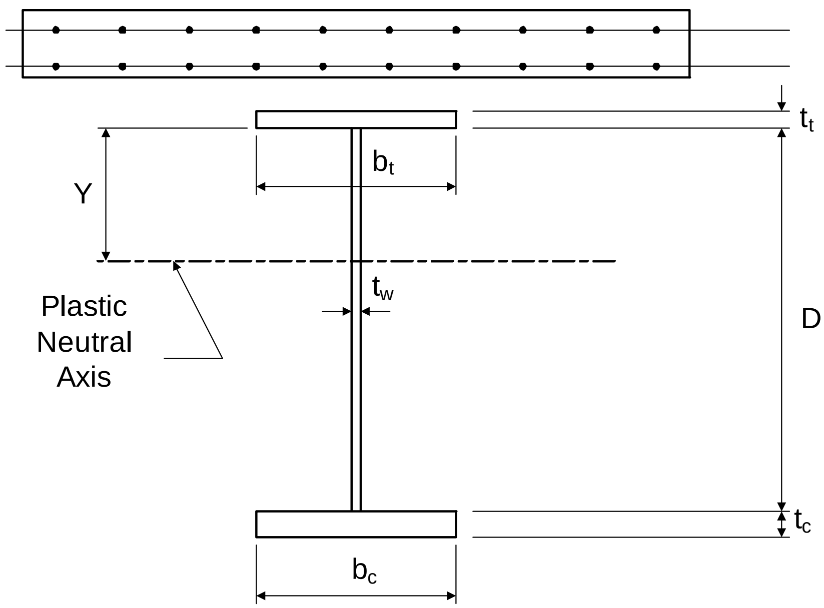

Figure E45-8.1-2
Composite Cross Section at Location of Maximum Negative Moment over Pier

$D := 54$ in

$t_W := 0.5$ in

E45-8.2 Compute Section Properties

Since the superstructure is composite, several sets of section properties must be computed. The initial dead loads (or the noncomposite dead loads) are applied to the girder-only section. For permanent loads assumed to be applied to the long-term composite section, the long-term modular ratio of 3n is used to transform the concrete deck area per **Std [10.35.1.4]**. For transient loads assumed applied to the short-term composite section, the short-term modular ratio of n is used to transform the concrete deck area.

The modular ratio, n, is for normal weight concrete is based upon $f`_c$ per **Std [10.38.1.3]**. For $f`_c$= 4,000 psi,

$$n := 8$$

For interior beams, the effective flange width is calculated the lesser of the following widths per **Std [10.38.3.1]**.

1. One-fourth the span length of the girder:

$$b_{eff1} := \frac{L}{4} \qquad \boxed{b_{eff1} = 30.00} \quad \text{ft}$$

2. The distance center to center of the girders:

$$b_{eff2} := S \qquad \boxed{b_{eff2} = 9.75} \quad \text{ft}$$

3. Twelve times the least thickness of the slab:

$$b_{eff3} := \frac{\left(12 \cdot t_s\right)}{12} \qquad \boxed{b_{eff3} = 8.50} \quad \text{ft}$$

Therefore, the effective flange width is:

$$b_{effflange} := \min\left(b_{eff1}, b_{eff2}, b_{eff3}\right) \qquad \boxed{b_{effflange} = 8.50} \quad \text{ft}$$

or

$$\boxed{b_{effflange} \cdot 12 = 102.00} \quad \text{in}$$

For this design example, the slab haunch is 3.5 inches throughout the length of the bridge. That is, the bottom of the slab is located 3.5 inches above the top of the web. The area of the haunch is conservatively not considered in the section properties for this example.

Based on the plate sizes shown in Figure E45-4.1-4, the noncomposite and composite section properties for the positive moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder.

The effect of creep from dead loads acting on the composite section shall be considered by checking stresses.

Positive Moment Region Section Properties						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	$A*y^2$ (Inches^4)	I_{total} (Inches^4)
Girder only:						
Top flange	10.500	55.250	580.1	0.5	8441.1	8441.6
Web	27.000	27.875	752.6	6561.0	25.8	6586.8
Bottom flange	12.250	0.438	5.4	0.8	8576.1	8576.9
Total	49.750	26.897	1338.1	6562.3	17043.0	23605.3
Composite (3n):						
Girder	49.750	26.897	1338.1	23605.3	11238.3	34843.6
Slab	36.125	62.625	2262.3	217.5	15477.0	15694.5
Total	85.875	41.926	3600.4	23822.8	26715.3	50538.0
Composite (n):						
Girder	49.750	26.897	1338.1	23605.3	29831.5	53436.8
Slab	108.375	62.625	6787.0	652.5	13694.3	14346.8
Total	158.125	51.384	8125.1	24257.8	43525.8	67783.6
Section	y_{botgdr} (Inches)	y_{topgdr} (Inches)	$y_{topslab}$ (Inches)	S_{botgdr} (Inches^3)	S_{topgdr} (Inches^3)	$S_{topslab}$ (Inches^3)
Girder only	26.897	28.728	---	877.6	821.7	---
Composite (3n)	41.926	13.699	24.949	1205.4	3689.3	2025.7
Composite (n)	51.384	4.241	15.491	1319.2	15982.9	4375.7

Table E45-8.2-1
Positive Moment Region Section Properties

Similarly, the noncomposite and composite section properties for the negative moment region are computed as shown in the following table. The distance to the centroid is measured from the bottom of the girder.

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not. However, per 45.6.3, only the top longitudinal mat of steel is used for rating purposes. With #6 bars at 7.5" o.c., the amount of longitudinal steel within the effective slab area is 5.98 in^2. Assume it is located 3 inches from the top of the slab. These values will be used for the calculations below.

Negative Moment Region Section Properties						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	A*y^2 (Inches^4)	I_{total} (Inches^4)
Girder only:						
Top flange	35.000	58.000	2030.0	18.2	30009.7	30027.9
Web	27.000	29.750	803.3	6561.0	28.7	6589.7
Bottom flange	38.500	1.375	52.9	24.3	28784.7	28809.0
Total	100.500	28.718	2886.2	6603.5	58823.1	65426.6
Composite (deck concrete using 3n):						
Girder	100.500	28.718	2886.2	65426.6	8995.9	74422.5
Slab	36.125	64.500	2330.1	217.5	25026.6	25244.1
Total	136.625	38.179	5216.3	65644.1	34022.5	99666.6
Composite (deck concrete using n):						
Girder	100.500	28.718	2886.2	65426.6	34639.7	100066.3
Slab	108.375	64.500	6990.2	652.5	32122.6	32775.1
Total	208.875	47.284	9876.4	66079.1	66762.3	132841.4
Composite (deck reinforcement only):						
Girder	100.500	28.718	2886.2	65426.6	435.2	65861.9
Deck reinf.	5.984	65.750	393.4	0.0	7309.8	7309.8
Total	106.484	30.799	3279.6	65426.6	7745.0	73171.6
Section	y_{botgdr} (Inches)	y_{topgdr} (Inches)	y_{deck} (Inches)	S_{botgdr} (Inches^3)	S_{topgdr} (Inches^3)	S_{deck} (Inches^3)
Girder only	28.718	30.532	---	2278.2	2142.9	---
Composite (3n)	38.179	21.071	30.571	2610.5	4730.1	3260.2
Composite (n)	47.284	11.966	21.466	2809.5	11101.3	6188.4
Composite (rebar)	30.799	28.451	34.951	2375.8	2571.9	2093.6

Table E45-8.2-2
Negative Moment Region Section Properties

E45-8.3 Dead Load Analysis - Interior Girder

Dead Load Components		
Resisted by	Type of Load Factor	
	DC	DW
Noncomposite section	• Steel girder • Concrete deck • Concrete haunch • Stay-in-place deck forms • Misc. (including cross-frames, stiffeners, etc.)	
Composite section	• Concrete parapets	• Future wearing surface & utilities

Table E45-8.3-1
Dead Load Components

COMPONENTS AND ATTACHMENTS: DC1 (NON-COMPOSITE)

GIRDER:

For the steel girder, the dead load per unit length varies due to the change in plate sizes. The moments and shears due to the weight of the steel girder can be computed using readily available analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

DECK:

For the concrete deck, the dead load per unit length for an interior girder is computed as follows:

$w_c = 0.150$ kcf

$S = 9.75$ ft

$t_{deck} = 9.00$ in

$$DL_{deck} := w_c \cdot S \cdot \frac{t_{deck}}{12} \qquad \boxed{DL_{deck} = 1.097} \quad \text{kip/ft}$$

HAUNCH:

For the concrete haunch, the dead load per unit length varies due to the change in top flange plate sizes. The moments and shears due to the weight of the concrete haunch can be computed using readily available analysis software. Since the top flange plate sizes are entered as input, the moments and shears due to the concrete haunch are computed based on the actual, varying haunch thickness.

MISC:

For the miscellaneous dead load (including cross-frames, stiffeners, and other miscellaneous structural steel), the dead load per unit length is assumed to be as follows (17.2.4.1):

$DL_{misc} := 0.030$ kip/ft

COMPONENTS AND ATTACHMENTS: DC2 (COMPOSITE)

PARAPET:

For the concrete parapets, the dead load per unit length is computed as follows, assuming that the superimposed dead load of the two parapets is distributed uniformly among all of the girders per **Std (3.23.2.3.1.1]**:

$w_{par} = 0.387$ kip/ft

$N_b = 5$

$$DL_{par} := \frac{w_{par} \cdot 2}{N_b} \qquad \boxed{DL_{par} = 0.155} \quad \text{kip/ft}$$

WEARING SURFACE: DW (COMPOSITE)

FUTURE WEARING SURFACE:

A future wearing surface of 20 psf will be used for the permit vehicle checks.

$$DW := \frac{0.020 \cdot w_{deck}}{N_b} \qquad \boxed{DW = 0.186} \quad \text{kip/ft}$$

Since the plate girder and its section properties are not uniform over the entire length of the bridge, analysis software was used to compute the dead load moments and shears.

The following two tables present the unfactored dead load moments and shears, as computed by an analysis computer program. Since the bridge is symmetrical, the moments and shears in Span 2 are symmetrical to those in Span 1.

Dead Load Moments (Kip-feet)											
Dead Load Component	**Location in Span 1**										
	0.0L	**0.1L**	**0.2L**	**0.3L**	**0.4L**	**0.5L**	**0.6L**	**0.7L**	**0.8L**	**0.9L**	**1.0L**
Steel girder	0.0	71.7	119.0	141.9	140.5	114.7	64.7	-10.1	-112.5	-247.1	-427.0
Concrete deck & haunches	0.0	475.4	787.6	936.9	923.0	746.1	406.2	-96.8	-765.9	-1592.1	-2584.3
Miscellaneous Steel Weight	0.0	12.6	18.6	24.8	24.5	19.8	10.8	-2.6	-20.2	-42.2	-68.5
Concrete parapets	0.0	66.7	111.2	133.3	135.7	110.7	66.0	-1.0	-90.3	-201.9	-335.8
Future wearing surface	0.0	75.9	126.4	151.6	151.4	125.9	75.0	-1.2	-102.7	-229.6	-381.8

Table 45E-8.3-2
Dead Load Moments

Dead Load Shears (Kips)											
Dead Load Component	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Steel girder	7.0	5.0	2.9	0.9	-1.1	-3.2	-5.2	-7.2	-9.8	-12.9	-17.0
Concrete deck & haunches	46.4	32.8	19.2	5.6	-8.0	-21.5	-35.1	-48.7	-62.3	-78.9	-89.5
Miscellaneous Steel Weight	1.2	0.9	0.5	0.2	-0.2	-0.6	-0.9	-1.3	-1.7	-2.0	-2.4
Concrete parapets	6.5	4.6	2.8	0.9	-0.9	-2.8	-4.7	-6.5	-8.4	-10.2	-12.1
Future wearing surface	7.4	5.3	3.2	1.0	-1.1	-3.2	-5.3	-7.4	-9.5	-11.6	-13.7

Table 45E-8.3-3
Dead Load Shears

E45-8.4 Compute Live Load Distribution Factors for Interior Girder

The live load distribution factors for an interior girder are computed as follows from **Std [3.23.2.2]**:

For one Design Lane Loaded:

$$DF_s := \frac{S}{7} \qquad \boxed{DF_s = 1.39} \text{ wheels}$$

For Two or More Design Lanes Loaded:

$$DF_m := \frac{S}{5.5} \qquad \boxed{DF_m = 1.77} \text{ wheels}$$

The live load impact percentage increase is calcuated per Std [3.8.2.1]:

$$IMPACT := 100 \cdot \min\left(0.3, \frac{L}{L + 125}\right) \qquad \boxed{IMPACT = 30.00} \%$$

From live load analysis software, the live load effects (per wheel including impact) are listed in the following table:

HS20 Live Load Effects (for Interior Beams)											
Live Load Effect	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Maximum positive moment (K-ft)	0.0	710.6	1190.6	1462.0	1564.4	1513.7	1335.0	1032.5	643.3	206.2	0.0
Maximum negative moment (K-ft)	0.0	-102.5	-205.0	-307.5	-410.0	-512.5	-615.1	-717.6	-823.1	-1264.2	-1967.9
Maximum positive shear (kips)	77.0	59.7	50.5	41.6	33.1	25.2	17.6	11.0	5.6	1.8	0.0
Maximum negative shear (kips)	-10.2	-10.3	-15.6	-22.6	-32.7	-42.6	-50.7	-57.8	-64.0	-71.9	-80.8

Table 45E-8.4-2
Live Load Effects

Two sections will be checked for illustrative purposes. First, the ratings will be performed for the location of maximum positive moment, which is at 0.4L in Span 1. Second, the ratings will be performed for the location of maximum negative moment and maximum shear, which is at th pier.

The following are for the location of maximum positive moment, which is at 0.4L in Span 1, as shown in Figure E45-8.4-1.

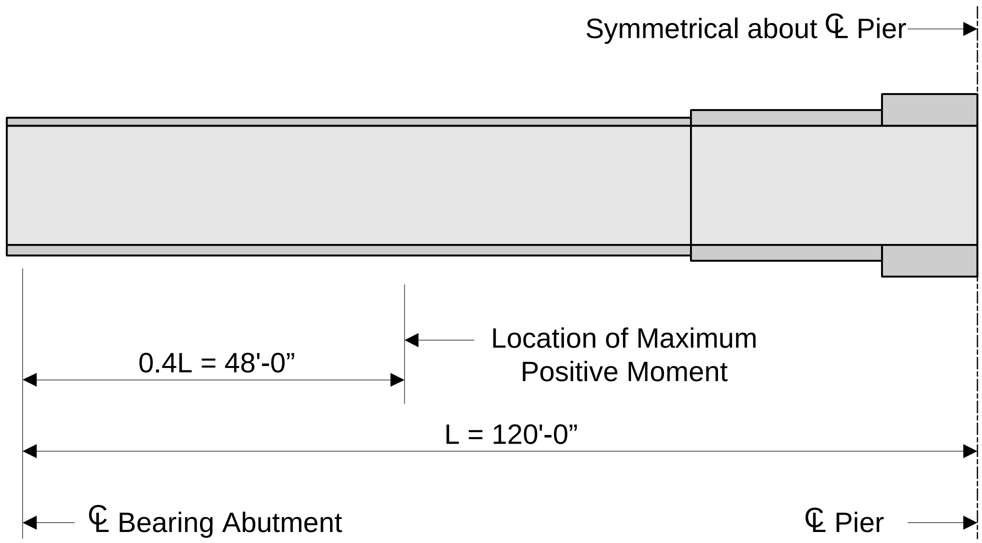

Figure E45-8.4-1
Location of Maximum Positive Moment

E45-8.5 Compute Plastic Moment Capacity - Positive Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis per **Std [10.50.1.1]**.

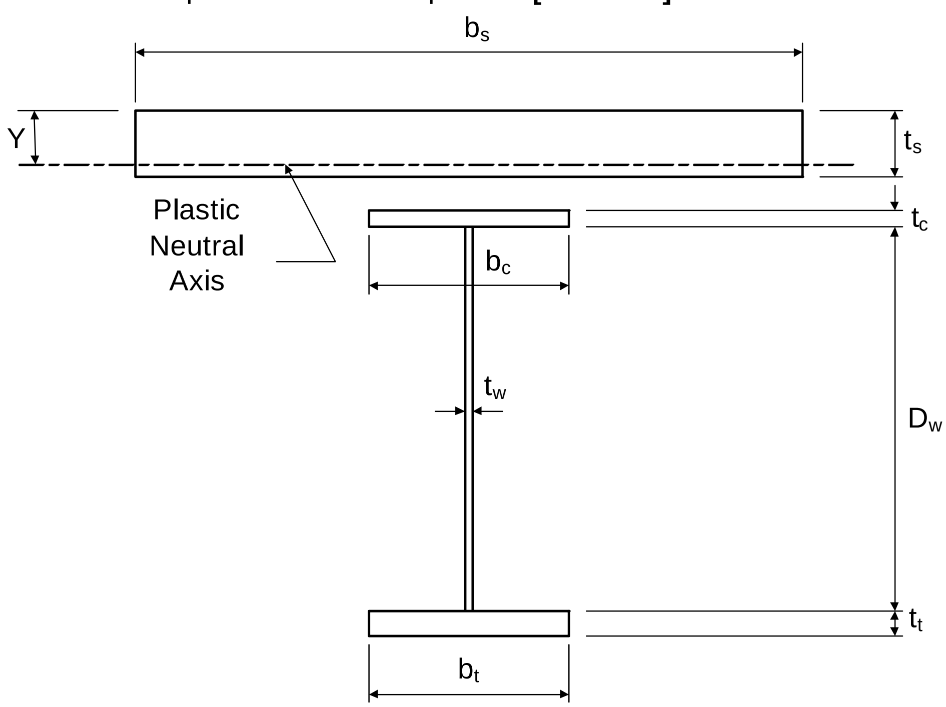

Figure E45-8.5-1
Computation of Plastic Moment Capacity for Positive Bending Sections

For the slab, the compressive force is equal to the smallest value given by the following equations:

$C_1 = 0.85 \cdot f'_c \cdot b_s \cdot t_s + (AF_y)_c$ **Std [Eq. 10-123]**

Where:

b_s = Effective width of concrete deck (in)

t_s = Thickness of concrete deck (in)

$f'_c = 4.00$ ksi

$b_s = 102.00$ in

$t_s = 8.50$ in

$(AF_y)_c$ is the product of the area and yield point of that part of reinforcement which lies in the compression zone of the slab. Neglecting this reinforcement contribution, the equation reduces to:

$C_1 := 0.85 \cdot f'_c \cdot b_s \cdot t_s$ $\boxed{C_1 = 2948}$ kips

$C_2 = (AF_y)_{bf} + (AF_y)_{tf} + (AF_y)_w$ **Std [Eq. 10-124]**

This equation reduces to equal the product of the girder steel area and its yield point:

$C_2 := (49.75) \cdot (50)$ $\boxed{C_2 = 2488}$ kips

The compressive force in the slab, C, is equal to:

$C := \min(C_1, C_2)$ $\boxed{C = 2488}$ kips

The depth of the stress block is computed from the compressive force in the slab:

$a := \dfrac{C}{0.85 \cdot f'_c \cdot b_s}$ **Std [Eq. 10-125]**

$\boxed{a = 7.17}$ in

Because C1 exceeds C2, the top portion of the steel section is not in compression. Therefore the plastic neutral axis (PNA) is located at the bottom of the concrete stress block, and no steel elements need to be checked for compactness. The plastic moment, M_p, is calculated using the force equilibrium method. The moment arm between the slab's compressive force and the PNA is equal to a/2, and the moment arm between the steel girder and the PNA is equal to 32.805 in.

$M_{p\_slab} := C \cdot \dfrac{a}{2} = 8921$ $\boxed{M_{p\_slab} = 8921}$ k-in

$M_{p\_girder} := C_2 \cdot 32.805$ $\boxed{M_{p\_girder} = 81602}$ k-in

$M_p := \dfrac{(M_{p\_slab} + M_{p\_girder})}{12}$ $\boxed{M_p = 7544}$ k-ft

In continuous spans with noncompact noncomposite or composite negative-moment pier sections, the maximum bending strength, M_n, of the composite positive-moment sections shall be taken as either the moment capacity at first yield or as:

$$M_n := M_y + A\cdot(M_{u\_pier} - M_{s\_pier})$$ **Std [Eq. 10-129d]**

Where:

M_y	=	the moment capacity at first yield of the compact positive moment section
$(M_{u\_pier} - M_{s\_pier})$	=	moment capacity of the noncompact section at the pier from **Std [10.48.2]** or **[10.48.4]** minus the elastic moment at the pier for the loading producing maximum positive bending in the span.
A	=	distance from end support to the location of maximum positive moment divided by the span length for end spans.

The moment capacity and first yield, M_y, is computed as follows, considering the application of the factored dead and live loads to the steel and composite sections:

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

Where:

M_{D1} = Bending moment caused by the factored permanent load applied before the concrete deck has hardened or is made composite (kip-in)

S_{NC} = Noncomposite elastic section modulus (in^3)

M_{D2} = Bending moment caused by the factored permanent load applied to the long-term composite section (kip-in)

S_{LT} = Long-term composite elastic section modulus (in^3)

M_{AD} = Additional bending moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange (kip-in)

S_{ST} = Short-term composite elastic section modulus (in^3)

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

$F_y := 50$ ksi

$M_{D1} := [1.3\cdot(M_{girder} + M_{deck} + M_{misc})]$ $\boxed{M_{D1} = 1414}$ kip-ft

$M_{D2} := (1.3\cdot M_{DC2})$ $\boxed{M_{D2} = 176}$ kip-ft

For the bottom flange:

$S_{NC\_pos} = 877.63$ in^3

$S_{LT\_pos} = 1205.40$ in^3

$S_{ST\_pos} = 1319.16$ in^3

$$M_{AD} := \left[\frac{S_{ST\_pos}}{12^3} \cdot \left(F_y \cdot 12^2 - \frac{M_{D1}}{\frac{S_{NC\_pos}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos}}{12^3}}\right)\right]$$

$\boxed{M_{AD} = 3177}$ kip-ft

$M_{ybot} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ybot} = 4768}$ kip-ft

For the top flange:

$S_{NC\_pos\_top} = 821.67$ in^3

$S_{LT\_pos\_top} = 3689.31$ in^3

$S_{ST\_pos\_top} = 15982.90$ in^3

$$M_{AD} := \frac{S_{ST\_pos\_top}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC\_pos\_top}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos\_top}}{12^3}}\right)$$

$\boxed{M_{AD} = 38319}$ kip-ft

$M_{ytop} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ytop} = 39910}$ kip-ft

The yield moment, M_y, is the lesser value computed for both flanges. Therefore, M_y is determined as follows:

$M_y := \min(M_{ybot}, M_{ytop})$ $\boxed{M_y = 4768}$ kip-ft

From calculations to follow for negative moment, moment capacity of the noncompact section a the pier is:

$M_{u\_pier} := 9899$ k-ft

From live load analysis software, the elastic moment at the pier for the loading producing maximum positive bending in the span is:

$M_{s\_pier} := 4431.52$ k-ft

The distance from end support to the location of maximum positive moment divided by the span length is:

$A := 0.4$

Therefore:

$M_n := M_y + A \cdot (M_{u\_pier} - M_{s\_pier})$ $\boxed{M_n = 6955}$ kip-ft

E45-8.6 Design Load Rating @ 0.4L

$$RF = \frac{M_n - A_1 \cdot M_{DL}}{A_2(M_{LLIM})}$$

Where:

$M_{DL} := M_{girder} + M_{deck} + M_{misc} + M_{DC2}$ $\boxed{M_{DL} = 1224}$ kip-ft

$M_{LLIM} := M_{LL}$ $\boxed{M_{LLIM} = 1564}$ kip-ft

Inventory

$$RF_{inv\_0.4L} := \frac{M_n - 1.3 \cdot M_{DL}}{2.17 \cdot (M_{LLIM})}$$ $\boxed{RF_{inv\_0.4L} = 1.58}$

Operating

$$RF_{op\_0.4L} := \frac{M_n - 1.3 \cdot M_{DL}}{1.3 \cdot (M_{LLIM})}$$ $\boxed{RF_{op\_0.4L} = 2.64}$

E45-8.7 Check Section Proportion Limits - Negative Moment Region

Now the specification checks are repeated for the location of maximum negative moment, whicl is at the pier, as shown in Figure 24E1.17-1. This is also the location of maximum shear in this case.

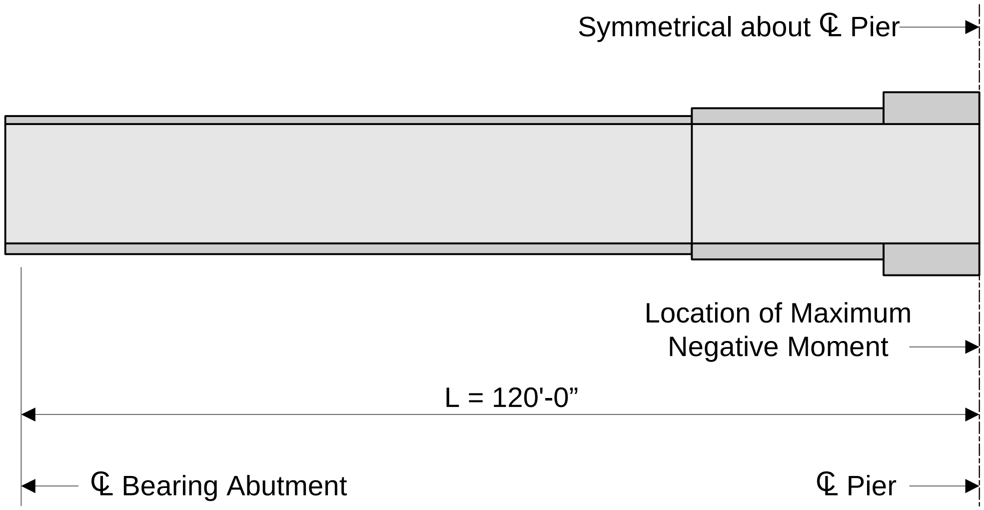

Figure E45-8.7-1
Location of Maximum Negative Moment

For a section to be compact, it must meet the proportion limits with **Std [10.48.1.1]**. For 50 ksi steel, these are as follows:

Compression Flange $\frac{b_f}{2 \cdot t_f} \le 18.4$ **Std [Eq. 10-93]**

$b_f := 14$

$t_f := 2.75$ $\boxed{\frac{b_f}{2 \cdot t_f} = 2.55}$ OK

Web Thickness $\frac{D}{t_w} \le 86$ **Std [Eq. 10-94]**

$D = 54.00$

$t_w = 0.50$ $\boxed{\frac{D}{t_w} = 108.00}$ FAILS

Therefore the section is noncompact at the pier. The requirements of Braced Noncompact Sections per **Std [10.48.2]** will be checked:

Compression Flange $\frac{b_f}{2 \cdot t_f} \le 24$ **Std [Eq. 10-100]**

$\boxed{\frac{b_f}{2 \cdot t_f} = 2.55}$ OK

Web Thickness $\frac{D}{t_w} \le 163$ **Std [Eq. 10-104]**

$\boxed{\frac{D}{t_w} = 108.00}$ OK

Lateral Bracing $L_b \le \frac{20000 \cdot A_f}{F_y \cdot d}$ **Std [Eq. 10-101]**

$A_f := (14)(2.75)$

$d := 54 + 2.75 + 2.5$

$L_b = 240.00$ $\boxed{\frac{20000 \cdot A_f}{F_y \cdot d} = 259.92}$ OK

E45-8.8 Compute Plastic Moment Capacity - Negative Moment Region

The negative moment capacity will be determined from **Std [10.50.2.2]** for noncompact negative moment sections.

Tension Flange $F_{ut} := F_y$

Compression Flange $F_{uc} := F_{cr} \cdot R_b$

$$F_{cr} := \frac{\left(4400 \cdot \frac{2t_f}{b_f}\right)^2}{1000} \le F_y$$

$$\frac{\left(4400 \cdot \frac{2t_f}{b_f}\right)^2}{1000} = 2987.96$$

Therefore $F_{cr} := F_y$ $\boxed{F_{cr} = 50.00}$ ksi

$R_b := 1.0$ due to adequate lateral bracing per **Std [Eq. 10-101]**

$F_{uc} := F_{cr} \cdot R_b = 50.00$ $\boxed{F_{uc} = 50.00}$ ksi

The moment capacity is taken as the lesser of the maximum strengths at the tension or compression flanges:

$S_{xt} := S_{rebar\_top}$ $\boxed{S_{xt} = 2572}$ in^3

$M_{u1} := F_y \cdot \frac{S_{xt}}{12}$ $\boxed{M_{u1} = 10716}$ kip-ft

$S_{xc} := S_{rebar}$ $\boxed{S_{xc} = 2376}$ in^3

$M_{u2} := F_{cr} \cdot R_b \cdot \frac{S_{xc}}{12}$ $\boxed{M_{u2} = 9899}$ kip-ft

$M_{n\_neg} := \min(M_{u1}, M_{u2})$ $\boxed{M_{n\_neg} = 9899}$ kip-ft

E45-8.9 Design Load Rating @ Pier

$$RF = \frac{M_{n\_neg} - A_1 \cdot M_{DL\_neg}}{A_2 (M_{LLIM\_neg})}$$

Where:

$M_{DL\_neg} := M_{girder\_neg} + M_{deck\_neg} + M_{misc\_neg} + M_{DC2\_neg}$

$\boxed{M_{DL\_neg} = -3415}$ kip-ft

$M_{LLIM\_neg} := M_{LL\_neg}$ $\boxed{M_{LLIM\_neg} = -1968}$ kip-ft

A. Steel Flexure Moment Strength **MBE [6B.4.1]**

$RF_{inv\_1.0L} := \frac{-M_{n\_neg} - 1.3 \cdot M_{DL\_neg}}{2.17 \cdot (M_{LLIM\_neg})}$ $\boxed{RF_{inv\_1.0L} = 1.28}$

$RF_{op\_1.0L} := \frac{-M_{n\_neg} - 1.3 \cdot M_{DL\_neg}}{1.3 \cdot (M_{LLIM\_neg})}$ $\boxed{RF_{op\_1.0L} = 2.13}$

E45-8.10 Rate for Shear - Negative Moment Region

Shear must be checked at each section of the girder. For this Rating example, shear is maximum at the pier, and will only be checked there for illustrative purposes.

The transverse intermediate stiffener spacing is 120". The spacing of the transverse intermediate stiffeners does not exceed 3D, therefore the section can be considered stiffened and the provisions of **Std [10.48.8]** apply.

$d_o := 120$ in

$D = 54.00$ in

$$k := 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \qquad \boxed{k = 6.01}$$

$$\frac{D}{t_w} = 108.00 \qquad \frac{D}{t_w} \geq 7500 \cdot \sqrt{\frac{k}{1000F_{yw}}} \qquad 7500 \cdot \sqrt{\frac{k}{1000F_{yw}}} = 82.24$$

$$C := \frac{4.5 \cdot 10^7 \cdot k}{\left(\frac{D}{t_w}\right)^2 \cdot \left(F_{yw} \cdot 1000\right)} = 0.46 \qquad \boxed{C = 0.464}$$

The plastic shear force, V_p, is then:

$$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w \qquad \boxed{V_p = 783.0} \text{ kips}$$

Std [Eq. 10-115]

$$V_n := V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \right] \qquad \boxed{V_n = 513.1} \text{ kips}$$

Std [Eq. 10-114]

HS-20 Maximum Shear @ Pier:

$$V_{DL} := V_{girder} + V_{deck} + V_{misc} + V_{DC2} \qquad \boxed{V_{DL} = -121.0} \text{ kips}$$

$V_{LL} = -80.75$ kips

E45-8.11 Design Load Rating @ Pier for Shear

$$RF = \frac{V_n - A_1 \cdot V_{DL}}{A_2 \cdot V_{LL}}$$

Strength Limit State

Inventory

$$RF_{inv\_shear} := \frac{-V_n - 1.3V_{DL}}{2.17 \cdot V_{LL}} \qquad \boxed{RF_{inv\_shear} = 2.03}$$

Operating

$$RF_{op\_shear} := \frac{-V_n - 1.3V_{DL}}{1.3 \cdot V_{LL}} \qquad \boxed{RF_{op\_shear} = 3.39}$$

Combined Moment and Shear **MBE [L6B2.3]**

$V_D := -V_{DL} = 120.97$

$V_L := -V_{LL} = 80.75$ kips

$V_n = 513.1$ kips

$V_p = 783.00$ $C = 0.46$

For a composite noncompact section, the initial moment rating factor shall be taken as the smaller of the rating factors determined separately for the compression and tension flange. Stresses (f_D, f_L) are substituted for moments (M_D, M_L).

$M_D := -M_{DL\_neg} = 3415$ kip-ft

$M_L := 1442.06$ Concurrent live load from analysis software

$$f_D := \max\left(\frac{12 \cdot M_D}{S_{xt}}, \frac{12 \cdot M_D}{S_{xc}}\right) \qquad \boxed{f_D = 17.25} \text{ ksi}$$

$$f_L := \max\left(\frac{12 \cdot M_L}{S_{xt}}, \frac{12 \cdot M_L}{S_{xc}}\right) \qquad \boxed{f_L = 7.28} \text{ ksi}$$

$F_n := F_y$

Step 1 - Determine initial rating factors ignoring interaction:

$RF_{v1\_inv} := RF_{inv\_shear}$ $\boxed{RF_{v1\_inv} = 2.03}$

$RF_{m1\_inv} := \dfrac{F_n - 1.3 \cdot f_D}{2.17 \cdot f_L}$ $\boxed{RF_{m1\_inv} = 1.74}$

$RF_{v1\_op} := RF_{op\_shear}$ $\boxed{RF_{v1\_op} = 3.39}$

$RF_{m1\_op} := \dfrac{F_n - 1.3 \cdot f_D}{1.3 \cdot f_L}$ $\boxed{RF_{m1\_op} = 2.91}$

Step 2 - Determine initial controlling rating factor ignoring interaction:

$RF_{mv1\_inv} := \min(RF_{v1\_inv}, RF_{m1\_inv})$ $\boxed{RF_{mv1\_inv} = 1.74}$

$RF_{mv1\_op} := \min(RF_{v1\_op}, RF_{m1\_op})$ $\boxed{RF_{mv1\_op} = 2.91}$

Step 3 - Determine the factored moment and shear using the initial controlling rating factor from Step 2 as follows:

$V_1 := 1.3 \cdot V_D + RF_{mv1\_inv} \cdot 2.17 \cdot V_L$ $\boxed{V_1 = 462.9}$ kips

$f_1 := 1.3 \cdot f_D + RF_{mv1\_inv} \cdot 2.17 \cdot f_L$ $\boxed{f_1 = 50.00}$ ksi

Step 4 - Determine the final controlling rating factor as follows:

$0.6V_n = 308$ $V_1 > 0.6V_n$

$0.75F_n = 37.5$ $f_1 > 0.75F_n$

CASE D applies:

$$RF_{mvf1\_inv} := \frac{2.2V_n \cdot F_n - 1.3 \cdot V_D \cdot F_n - 1.6 \cdot 1.3 \cdot f_D \cdot V_n}{2.17 \cdot V_L \cdot F_n + 1.6 \cdot 2.17 \cdot f_L \cdot V_n} = 1.39$$

$$> \quad \frac{C \cdot V_p - 1.3V_D}{2.17 \cdot V_L} = 1.18$$

$$RF_{mvf1\_op} := \frac{2.2V_n \cdot F_n - 1.3 \cdot V_D \cdot F_n - 1.6 \cdot 1.3 \cdot f_D \cdot V_n}{1.3 \cdot V_L \cdot F_n + 1.6 \cdot 1.3 \cdot f_L \cdot V_n} = 2.32$$

$$> \quad \frac{C \cdot V_p - 1.3V_D}{1.3 \cdot V_L} = 1.96$$

Therefore

$RF_{vf1\_inv} := RF_{mvf1\_inv}$ $\boxed{RF_{vf1\_inv} = 1.39}$

$RF_{mf1\_inv} := RF_{mvf1\_inv}$ $\boxed{RF_{mf1\_inv} = 1.39}$

$RF_{vf1\_op} := RF_{mvf1\_op}$ $\boxed{RF_{vf1\_op} = 2.32}$

$RF_{mf1\_op} := RF_{mvf1\_op}$ $\boxed{RF_{mf1\_op} = 2.32}$

Step 5 - If the controlling RF is different than the initial controlling RF, repeat Steps 2-4 (using the final controlling RF as the initial controlling RF):

$RF_{mv2\_inv} := \min(RF_{vf1\_inv}, RF_{mf1\_inv})$ $RF_{mv2\_inv} = 1.39$

$V_2 := 1.3 \cdot V_D + RF_{mv2\_inv} \cdot 2.17 \cdot V_L$ $\boxed{V_2 = 400.4}$ kips

$V_2 > 0.6V_n$

$f_2 := 1.3 \cdot f_D + RF_{mv2\_inv} \cdot 2.17 \cdot f_L$ $\boxed{f_2 = 44.36}$ ksi

$M_2 > 0.75M_{n\_neg}$

CASE D applies again, so the calculation does not need to be repeated.

$RF_{mvf\_inv} := RF_{mf1\_inv}$ $\boxed{RF_{mvf\_inv} = 1.39}$

$RF_{mvf\_op} := RF_{mf1\_op}$ $\boxed{RF_{mvf\_op} = 2.32}$

Since RF>1.20 @ operating for all checks, posting vehicle checks are not required for this example.

E45-8.12 - Permit Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.12).

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming full dynamic load allowance is utilized. Future wearing surface shall not be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW. Future wearing surface shall be included in the check.

E45-8.12.1 - Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

The values from this analysis are used for performing the Wis-SPV design check per 45.12

Load Distribution Factors

Single Lane Interior DF $\boxed{DF_S = 1.39}$

Wis-SPV Moments and Shears from LL analysis software, with impact and distribution factors included:

$M_{LL\_0.4L} := 2393.45$ kip-ft

$M_{LL\_1.0L} := 1836.47$ kip-ft

$V_{LL\_1.0L} := 132.47$ kips

The DL moments and shears with wearing surface included are:

$M_{DL\_0.4L} := M_{girder} + M_{deck} + M_{misc} + M_{DC2} + M_{DW}$

$\boxed{M_{DL\_0.4L} = 1379}$ kip-ft

$M_{DL\_1.0L} := -\left(M_{girder\_neg} + M_{deck\_neg} + M_{misc\_neg} + M_{DC2\_neg} + M_{DW\_neg}\right)$

$\boxed{M_{DL\_1.0L} = 3787}$ kip-ft

$V_{DL\_1.0L} := -\left(V_{girder} + V_{deck} + V_{misc} + V_{DC2} + V_{DW}\right)$

$\boxed{V_{DL\_1.0L} = 134.7}$ kips

In continuous spans with noncompact noncomposite or composite negative-moment pier sections, the maximum bending strength, M_n, of the composite positive-moment sections shall be taken as either the moment capacity at first yield or as:

$M_n := M_y + A \cdot \left(M_{u\_pier} - M_{s\_pier}\right)$ **Std [Eq. 10-129d]**

Where:

M_y	=	the moment capacity at first yield of the compact positive moment section
$(M_{u\_pier} - M_{s\_pier})$	=	moment capacity of the noncompact section at the pier from [10.48.2] or [10.48.4] minus the elastic moment at the pier for the loading producing maximum positive bending in the span.
A	=	distance from end support to the location of maximum positive moment divided by the span length for end spans.

The moment capacity and first yield, M_y, is computed as follows, considering the application of the factored dead and live loads to the steel and composite sections:

$$F_y = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

Where:

M_{D1} = Bending moment caused by the factored permanent load applied before the concrete deck has hardened or is made composite (kip-in)

S_{NC} = Noncomposite elastic section modulus (in^3)

M_{D2} = Bending moment caused by the factored permanent load applied to the long-term composite section (kip-in)

S_{LT} = Long-term composite elastic section modulus (in^3)

M_{AD} = Additional bending moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange (kip-in)

S_{ST} = Short-term composite elastic section modulus (in^3)

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

$F_y := 50$ ksi

$M_{D1} := 1.3 \cdot (M_{girder} + M_{deck} + M_{misc})$ $\boxed{M_{D1} = 1414}$ kip-ft

$M_{D2} := 1.3 \cdot (M_{DC2} + M_{DW})$ $\boxed{M_{D2} = 378}$ kip-ft

For the bottom flange:

$S_{NC\_pos} = 877.63$ in^3

$S_{LT\_pos} = 1205.40$ in^3

$S_{ST\_pos} = 1319.16$ in^3

$$M_{AD} := \left[\frac{S_{ST\_pos}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC\_pos}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos}}{12^3}}\right)\right]$$ $\boxed{M_{AD} = 2956}$ kip-ft

$M_{ybot} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ybot} = 4749}$ kip-ft

For the top flange:

$S_{NC\_pos\_top} = 821.67$ in^3

$S_{LT\_pos\_top} = 3689.31$ in^3

$S_{ST\_pos\_top} = 15982.90$ in^3

$$M_{AD} := \frac{S_{ST\_pos\_top}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC\_pos\_top}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT\_pos\_top}}{12^3}} \right) \qquad \boxed{M_{AD} = 37444} \text{ kip-ft}$$

$$M_{ytop} := M_{D1} + M_{D2} + M_{AD} \qquad \boxed{M_{ytop} = 39237} \text{ kip-ft}$$

The yield moment, M_y, is the lesser value computed for both flanges. Therefore, M_y is determined as follows:

$$M_y := \min(M_{ybot}, M_{ytop}) \qquad \boxed{M_y = 4749} \text{ kip-ft}$$

The moment capacity of the noncompact section at the pier is:

$$M_{u\_pier} := M_{n\_neg} \qquad \boxed{M_{u\_pier} = 9899} \text{ kip-ft}$$

From live load analysis software, the elastic moment at the pier for the loading producing maximum positive bending in the span is:

$M_{s\_pier} := 4918.05$ kip-ft

The distance from end support to the location of maximum positive moment divided by the span length is:

$A := 0.4$

Therefore:

$$M_{n\_spv} := M_y + A \cdot (M_{u\_pier} - M_{s\_pier}) \qquad \boxed{M_{n\_spv} = 6742} \text{ kip-ft}$$

At the pier, the flexural and shear capacity are equal to the values calculated for the HS20 load:

$M_{n\_neg} = 9899$ kip-ft

$V_n = 513.1$ kips

The operating-level rating factors may then be calculated as:

$$RF_{pos} := \frac{M_{n\_spv} - 1.3 \cdot M_{DL\_0.4L}}{1.3 \cdot M_{LL\_0.4L}}$$ $\boxed{RF_{pos} = 1.59}$

$\boxed{RF_{pos} \cdot 190 = 302.2}$ kips

$$RF_{neg} := \frac{M_{n\_neg} - 1.3 \cdot M_{DL\_1.0L}}{1.3 \cdot M_{LL\_1.0L}}$$ $\boxed{RF_{neg} = 2.08}$

$\boxed{RF_{neg} \cdot 190 = 396.0}$ kips

$$RF_{shear} := \frac{V_n - 1.3 \cdot V_{DL\_1.0L}}{1.3 \cdot V_{LL\_1.0L}}$$ $\boxed{RF_{shear} = 1.96}$

$\boxed{RF_{shear} \cdot 190 = 373.0}$ kips

Combined Moment and Shear at Pier **MBE [L6B2.3]**

$V_D := V_{DL\_1.0L} = 134.7$ kips

$V_L := V_{LL\_1.0L} = 132.5$ kips

$V_n = 513.1$ kips

$V_p = 783.0$ kips $C = 0.46$

For a composite noncompact section, the initial moment rating factor shall be taken as the smaller of the rating factors determined separately for the compression and tension flange. Stresses (f_D, f_L) are substituted for moments (M_D, M_L).

$M_D := M_{DL\_1.0L} = 3787$ kip-ft

$M_L := 1318.04$ kip-ft Concurrent single-lane Wis-SPV live load from analysis software

$$f_D := \max\left(\frac{12 \cdot M_D}{S_{xt}}, \frac{12 \cdot M_D}{S_{xc}}\right)$$ $\boxed{f_D = 19.13}$ ksi

$$f_L := \max\left(\frac{12 \cdot M_L}{S_{xt}}, \frac{12 \cdot M_L}{S_{xc}}\right)$$ $\boxed{f_L = 6.66}$ ksi

$F_n := F_y$

Step 1 - Determine initial rating factors ignoring interaction:

$RF_{v1\_op} := RF_{shear}$ $\boxed{RF_{v1\_op} = 1.96}$

$RF_{neg} := \dfrac{F_n - 1.3 \cdot f_D}{1.3 \cdot f_L}$ $\boxed{RF_{neg} = 2.90}$

Step 2 - Determine initial controlling rating factor ignoring interaction:

$RF_{mv1\_op} := \min\left(RF_{v1\_op}, RF_{m1\_op}\right)$ $\boxed{RF_{mv1\_op} = 1.96}$

Step 3 - Determine the factored moment and shear using the initial controlling rating factor from Step 2 as follows:

$V_1 := 1.3 \cdot V_D + RF_{shear} \cdot 1.3 \cdot V_L = 513.11$ $\boxed{V_1 = 513.1}$ kips

$f_1 := 1.3 \cdot f_D + RF_{neg} \cdot 1.3 \cdot f_L = 50.00$ $\boxed{f_1 = 50.00}$ ksi

Step 4 - Determine the final controlling rating factor as follows:

$0.6V_n = 308$ kips $V_1 > 0.6V_n$

$0.75F_n = 37.5$ kips $f_1 > 0.75F_n$

CASE D applies:

$$RF_{mvf1\_op} := \frac{2.2V_n \cdot F_n - 1.3 \cdot V_D \cdot F_n - 1.6 \cdot 1.3 \cdot f_D \cdot V_n}{1.3 \cdot V_L \cdot F_n + 1.6 \cdot 1.3 \cdot f_L \cdot V_n} = 1.74$$

$$> \frac{C \cdot V_p - 1.3V_D}{1.3 \cdot V_L} = 1.09$$

Therefore

$RF_{vf1\_op} := RF_{mvf1\_op}$ $\boxed{RF_{vf1\_op} = 1.74}$

$RF_{mf1\_op} := RF_{mvf1\_op}$ $\boxed{RF_{mf1\_op} = 1.74}$

Step 5 - If the controlling RF is different than the initial controlling RF, repeat Steps 2-4 (using the final controlling RF as the initial controlling RF):

$RF_{mv2\_op} := \min\left(RF_{vf1\_op}, RF_{mf1\_op}\right) = 1.74$ $\boxed{RF_{mv2\_op} = 1.74}$

$V_2 := 1.3 \cdot V_D + RF_{mv2\_op} \cdot 1.3 \cdot V_L = 473.91$ $\boxed{V_2 = 473.9}$ kips

$V_2 > 0.6V_n$

$f_2 := 1.3 \cdot f_D + RF_{mv2\_op} \cdot 1.3 \cdot f_L = 39.89$ $\boxed{f_2 = 39.89}$ ksi

$M_2 > 0.75M_{n\_neg}$

CASE D applies again, so the calculation does not need to be repeated.

$RF_{mvf\_op} := RF_{mf1\_op}$ $\boxed{RF_{mvf\_op} = 1.74}$

$\boxed{RF_{mvf\_op} \cdot 190 = 329.7}$ kips

Flexure at Positive Moment Controls

> 190k minimum : CHECK OK

E45-8.12.2 - Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

For use with plans and rating sheet only.

By inspection, since the governing limit state and location for the single-lane Wis-SPV w/ FWS was positive moment at 0.4L, it will be the same for the single-lane Wis-SPV w/o FWS.

The positive moment capacity which is based upon M_y and M_s needs to be recalculated.

$M_y := 4768$ kip-ft from HS20 calculation w/o FWS

From live load analysis software, the elastic moment at the pier for the loading producing maximum positive bending in the span is:

$M_{s\_pier} := 4431.52$ kip-ft

Therefore:

$M_{n\_spv} := M_y + A \cdot \left(M_{u\_pier} - M_{s\_pier}\right)$ $\boxed{M_{n\_spv} = 6955}$ kip-ft

$M_{DL\_0.4L} := M_{DL\_0.4L} - M_{DW}$ $\boxed{M_{DL\_0.4L} = 1224}$ kip-ft

$RF_{pos} := \frac{M_{n\_spv} - 1.3 \cdot M_{DL\_0.4L}}{1.3 \cdot M_{LL\_0.4L}}$ $\boxed{RF_{pos} = 1.72}$

$\boxed{RF_{pos} \cdot 190 = 327.6}$ kips

E45-8.12.3 - Wis-SPV Permit Rating with Multi-Lane Distribution

The multi-lane SPV check is calculated w/o future wearing surface. The governing location and the flexural capacity are equal to the results from the single-lane analysis. From live load analysis software, the maximum moment at 0.4L is:

$M_{LL\_0.4L} := 3046.21$ kip-ft

$RF_{pos} := \frac{M_{n\_spv} - 1.3 \cdot M_{DL\_0.4L}}{1.3 \cdot M_{LL\_0.4L}}$ $\boxed{RF_{pos} = 1.35}$

$\boxed{RF_{pos} \cdot 190 = 257.4}$ kips

E45-8.13 Summary of Rating

Steel Interior Girder					
Limit State	Design Load Rating		Wis-SPV Ratings (kips)		
	Inventory	Operating	Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Flexure @ 0.4L	HS 31	HS 52	302	327	257
Flexure @ 1.0L	HS 25	HS 42	396	N/A	N/A
Shear @ 1.0L	HS 40	HS 67	373	N/A	N/A
Combined Shear & Flexure @ 1.0L	HS 27	HS 46	329	N/A	N/A