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2.1 Organizational Charts

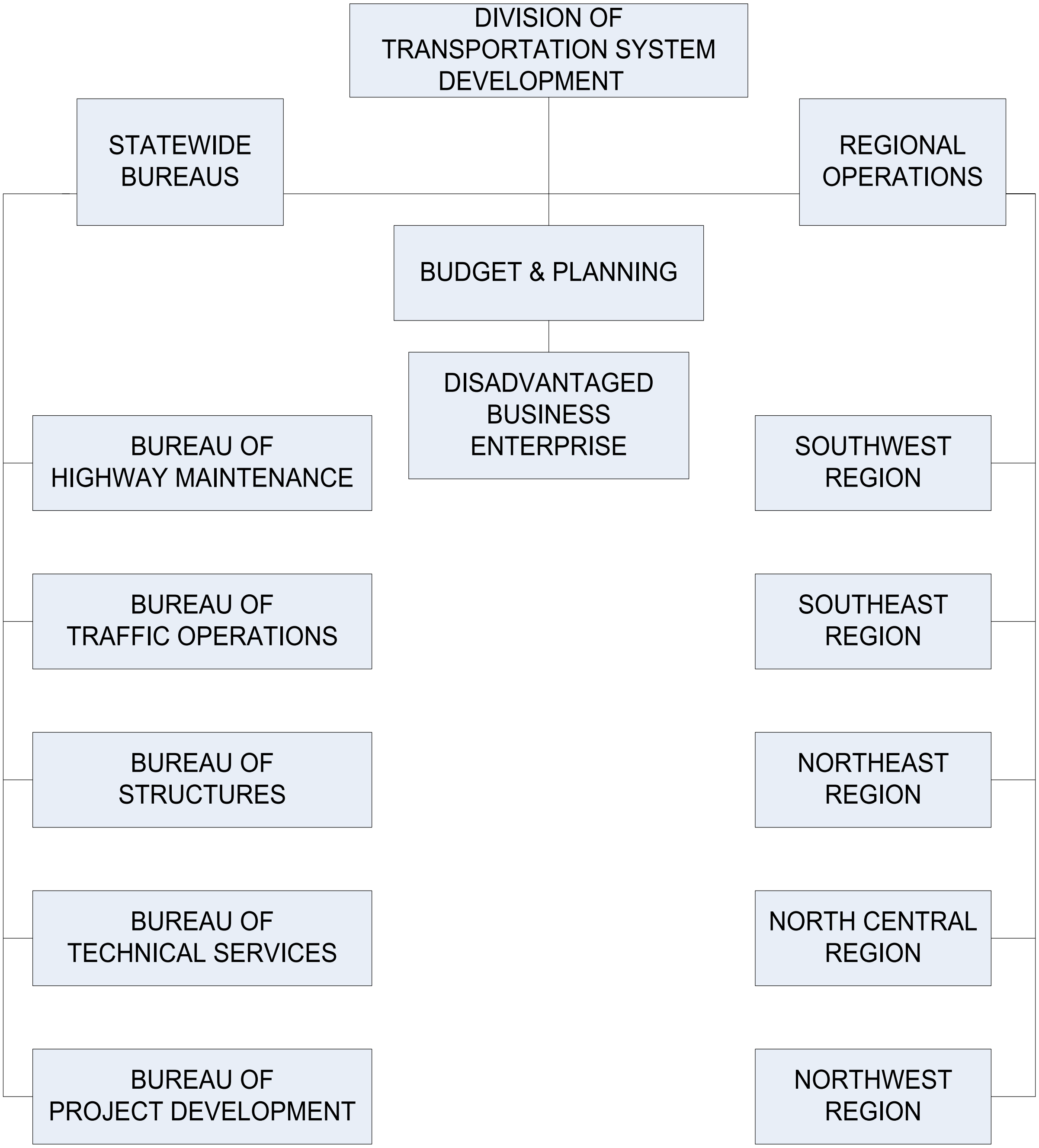


Figure 2.1-1
Division of Transportation System Development

2.5 Structure Numbers

An official number, referred to as a structure number, is assigned to bridge structures and ancillary structures in the WisDOT right-of-way. As shown in Figure 2.5-1, structure numbers begin with a letter based on the structure type. The structure type designation is then followed by a two-digit county number, a unique four-digit structure number, and in some cases a unit number. Note: leading zeroes may be omitted from the structure number (i.e. B-5-70).

Structure numbers should be assigned to structures prior to submitting information to the Bureau of Structures for the structural design process or the plan review process. For assigning structure numbers and structure unit numbers, contact the Regional Structures Program Manager for B-Structures and the Regional Ancillary Program Manager for ancillary structures. As of 2024, the practice of assigning unit numbers to bridge structures has been discontinued. Existing bridge structures assigned unit numbers will remain in place, unless directed otherwise. Refer to the WisDOT Structures Maintenance and Inspection website for additional information.

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.

The following criteria should be used when assigning structure numbers to bridge (B) and ancillary structures (C, P, S, L, R, N, or M):

- B is assigned to bridge structures (B-Structures) over 20 ft. in structure length, measured along the roadway centerline between the inside faces of abutments or exterior walls. A set of nested pipes may be assigned as a bridge structure if the distance between the inside diameters of the end pipes exceeds 20 ft. and the clear distance between pipe openings is less than half the diameter of the smallest pipe. Refer to the Structure Inspection Manual for measurements used to define a bridge structure. Bridges on state boundary lines also have a number designated by the adjacent state.

  Pedestrian only bridge structures are assigned a B-Structure if they are over 20 ft in structure length and are state maintained, DNR bridges reviewed by WisDOT, or cross a roadway. Pedestrian boardwalks may be assigned a B-Structure when a clear span exceeds 20 ft. Other cases may be considered on a project-to-project basis.

- In general, C is assigned to small bridge structures (C-Structures) 20 ft. or less in structure length that have a unique structural design and/or a heightened inspection interest. This includes bridge-like structures (deck girders, flat slabs, etc.), concrete box culverts with a cross-sectional opening greater than, or equal to 20 square feet, rigid frames (three-sided concrete structures), and structural plate structures (pipes, pipe arches, box culverts, etc.). Structures not meeting the bridge structure or small bridge structure criteria are then typically considered a roadway culvert as described in Facilities Development Manual (FDM) 13-1. Buried structures listed in FDM 13-1 are typically not assigned a structure number, except for closely nested pipes and structural plate structures. Refer to the Structure Inspection Manual for additional information on small bridge structures.

- P designates structures for which there are no structural plans on file.

WisDOT Policy Item:

No new P numbers will be assigned as we should always request plans.

- S is assigned to overhead sign structures and signal monotubes. Unit numbers should be assigned to signal monotubes at an intersection with multiple structures. In this case, the base structure number should be the same for all signal monotubes and the unit numbers use to designate individual structures (i.e. S-13-1421-0001, S-13-1421-0002, etc.).

- L is assigned to high mast lighting structures.

- R is assigned to permanent retaining walls. For a continuous wall consisting of various wall types, such as a secant pile wall followed by a soldier pile wall, unit numbers should be assigned to each wall type segment. Wall facing discontinuities (e.g. stairwells, staged construction, tiers, or changes to external loads) do not require unique wall numbers if the leveling pad or footing is continuous between the completed wall segments. For soldier pile walls with anchored and non-anchored segments, unique wall numbers are not required for each segment.

  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.

  Retaining walls whose height exceeds the below criteria require R numbers:

  - Proprietary retaining walls (e.g., modular block MSE walls)

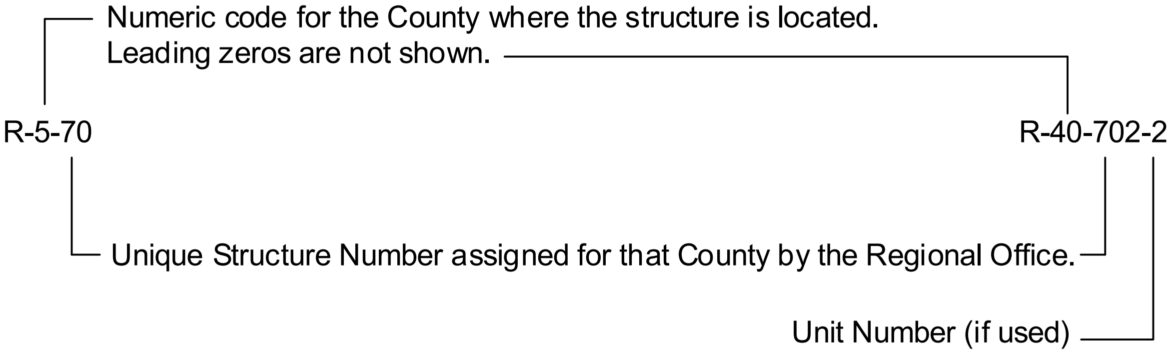
    - MSE walls having a maximum height of less than 5.5 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be “minor retaining walls” and do not require an R number. Refer to FDM 11-55-5.2 for more information.

    - 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, cast-in-place 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.

- N is assigned to noise barriers. Unit numbers may be assigned to long bridges or complex interchanges where it is desirable to have only one structure number for the site.
- M is assigned to miscellaneous structures where it is desirable to have a structure plan record while not meeting the above-mentioned structure assigned criteria.



<u>Figure 2.5-1</u>
Structure Number Detail

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

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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. For continuous girders, show and dimension tension zones on top and bottom flanges. Show the tension zones on replacement and rehabilitation projects as applicable, including on deck replacements projects, widenings, and overlay projects with substantial full-depth deck repair areas. See

Chapter 24 – Steel Girder Structures for additional information on tension zones. 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, Name plate location, Benchmark location, and 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 Benchmarks

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.

A benchmark location shall be shown on bridge and larger culvert plans. Locate the benchmark on a horizontal surface flush with the concrete and in close proximity to the name plate. When possible, locate on top of the parapet on the bridge deck, above the abutment. Do not locate benchmarks at locations where elevations are subject to movement (e.g. midspan) and avoid placing below a rail or fence system. Benchmarks are typically metal survey disks, which are to be supplied by the department and set by the contractor. See FDM 9-25-5 for additional benchmark information.

6.3.3.8 Removing Structure and Debris Containment

This section provides guidance for selecting the appropriate Removing Structure bid item and determining when to use the “Debris Containment” bid item.

The “Removing Structure (structure)” bid item is most typically used for complete or substantial removals, as described in 6.3.3.8.2, of grade separation structures and box culverts. In addition to this Standard Specification bid item, there are three additional Standard Specification bid items for complete or substantial removal work over waterways: “Removing Structure Over

Waterway Remove Debris (structure)"; "Removing Structure Over Waterway Minimal Debris (structure)"; and "Removing Structure Over Waterway Debris Capture (structure)". If these four Standard Specification bid items do not encapsulate site specific constraints for specialized cases, which should be a rare occurrence, the designer can utilize special provisions to augment the standard spec removal items.

The designer should review all of these Standard Specifications and coordinate with the Wisconsin Department of Natural Resources (DNR) to determine which bid items to use when removing a particular structure. **If the designer disagrees with the recommendation from the DNR's Initial Review Letter (IRL), the designer should work with WisDOT Regional Environmental Coordinator (REC), WisDOT Regional Stormwater & Erosion Control Engineer (SWECE) and DNR Transportation Liaison (TL) to come to a consensus on the appropriate bid item, considering constructability and cost impacts of the items.** For unique or difficult removals, designers should consult with the contracting community to assess costs and the feasibility of a particular removal technique. One of the following Removing Structure bid items should be selected for removals over waterways:

- Removing Structure Over Waterway <u>Remove Debris (structure)</u> is used where it is not possible to remove the structure without dropping it, or a portion of it, into a waterway or wetland; and that waterway or wetland is not highly environmentally sensitive. This bid item is typically appropriate for removing the following structure types: slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges.

- Removing Structure Over Waterway <u>Minimal Debris (structure)</u> is used where it is possible to remove the structure with only minimal debris dropping into a waterway or wetland, and that waterway or wetland is not highly environmentally sensitive. This bid item is typically appropriate for removing all structure types except for the following bridges which are typically covered under Removing Structure Over Waterway Remove Debris (structure): slab spans; voided slabs; cast-in-place concrete girder bridges; earth-filled bridges; large trestle bridges. This bid item will likely be used for most stream crossing removals. The designer may need to expand the standard spec with special provision language to address additional DNR concerns and/or issues. CMM 645.6 contains example removal and clean-up methods corresponding to this bid item.

- Removing Structure Over Waterway <u>Debris Capture (structure)</u> is typically used when resources are present such that additional protection is required due to the waterway or wetland being highly environmentally sensitive. Before including this bid item in the contract, consult with the DNR and the department's regional environmental coordinator, as well as BOS, to determine if this bid item is appropriate. The designer may need to expand the standard spec with special provision language to address pier or abutment removal, and other project specific details.

Debris Containment bid items are used where structure removal, reconstruction, or other construction operations may generate falling debris that might pose a safety hazard or environmental/contamination concern to facilities located under the structure. Two standard spec bid items for debris containment are available for use depending on the project location. For grade separation structures, "Debris Containment (structure)" is utilized. This item is most typically used where the removal area is located over a railroad, but may also be used over roadways, bike paths, pedestrian ways, or other facilities that will not be closed during removal operations.

The "Debris Containment Over Waterway (structure)" item is not used when one of the three Removing Structure Over Waterway standard spec bid items is used. This item may be used for structure repair projects occur over waterways where full removals are not involved. One example of this is a standalone joint replacement project at a stream crossing structure.

6.3.3.8.1 Structure Repairs

Structure repair work could include, but is not limited to, the following bid items:

- Removing Concrete Masonry Deck Overlay
- Removing Asphaltic Concrete Deck Overlay
- Removing Polymer Overlay
- Cleaning Parapets
- Cleaning Concrete Surfaces
- Cleaning Decks to Reapply Concrete Masonry Overlay
- Preparation Decks (type)
- Cleaning Decks
- Joint Repair
- Curb Repair
- Concrete Surface Repair
- Full-Depth Deck Repair

Removal work limited to the above items is already included in the respective bid item specification, therefore a Removing Structure bid item not required. Use of Debris Containment should be reviewed for the following conditions:

- For work **over waterways**, a method of protecting the waterway is needed in some cases. Use Debris Containment over Waterway (structure), **only as needed** based on the extent and location of removal, and environmental sensitivity of the waterway. Debris is expected to be minimal.

- For work **over roadways**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, so generally no additional specifications are needed. **It is expected that pertinent lanes of the underpass roadway are closed when falling debris is expected from above.** No additional specifications are needed unless specifically requested with sufficient reason, in which case use Debris Containment (structure) **only as needed**, based on the extent and location of removal. Debris is expected to be minimal.

- For work **over railroads**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, so generally no additional specifications are needed. Exception: containment of debris is required where Full-Depth Deck Repair is expected. Use Debris Containment (structure) if Full-Depth Deck Repair is expected, or **only as needed**, based on the extent and location of removal. Debris is expected to be minimal.

6.3.3.8.2 Complete or Substantial Removals

Complete or substantial removals, not covered by one of the bid items listed in 6.3.3.8.1, should use a Removing Structure bid item. Substantial removals could include, but are not limited to; decks, parapets, and wingwalls. The appropriate Removing Structure bid item should be selected and the need for a Debris Containment bid item should be reviewed for the following conditions:

- For work **over waterways or wetlands**, a method of protecting the waterway is needed if the removal area is located over the waterway. If the removal area is located over the waterway, use one of the three Removing Structure Over Waterway (structure) bid items noted in 6.3.3.8. If the removal area is not located over the waterway, use Removing Structure (structure). The Debris Containment Over Waterway (structure) item is not used for this work.

- For work **over roadways**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, and Standard Specification, Section 203 Removing Old Culverts and Bridges addresses removal. **It is expected that pertinent lanes of the underpass roadway are closed when falling debris is expected from above.** Use Removing Structure (structure). No additional specifications are needed unless specifically requested with sufficient reasoning. Use Debris Containment (structure) **only as needed**, based on the significance of the roadway and/or location of removal.

- For work **over railroads**, Standard Specification, Sections 104 and 107, addresses safety of the traveling public and damage to all property, and Standard Specification, Section 203 Removing Old Culverts and Bridges addresses removal. A method of protecting the railroad is needed if the removal area is located over the railroad. Use Removing Structure (structure). Use Debris Containment (structure) if the **removal area is located over the railroad, or only as needed**, based on the extent and location of removal.

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 Computations 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

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 19-5-3.2 for bid item usage guidance and quantity calculation guidance. Bid as LB and round to the nearest 5 lbs.

6.4.42 Asphaltic Overlays

Estimate the overlay quantity by using the theoretical average overlay thickness and add ½” for variations in the deck surface. Provide this average thickness on the plan, as well. Use 110 lbs/(square yard - inch) to calculate hot mix asphalt (HMA) and polymer modified asphalt (PMA) overlay quantities.

For HMA overlays use 0.07 gallons/square yard to calculate tack coat quantity, unless directed otherwise.

Coordinate asphaltic quantity assumptions with the Region and roadway designers.

6.4.43 Longitudinal Grooving

This quantity is typically used for High Performance Concrete (HPC) structures with a design speed of 40 mph or greater. See 17.8.2 for additional guidance. Record this quantity to the nearest square foot.

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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 (Q100) or 1% chance 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 Facilities Development Manual (FDM) 20-5-15.

8.1.3.3 DOT Facilities Development Manual

Refer to FDM Chapter 10 – Erosion Control and Storm Water Quality, FDM Chapter 11 – Design, FDM Chapter 13 - Drainage, and FDM Chapter 20 - 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 criterion 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 superstructure replacement provided that the substructure and berm configuration remain unchanged, and the low chord elevation is not significantly lowered.

The designer should consider historical high-water elevations, Flood Insurance Studies and inundation potential 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 *Estimating Flood Magnitude and Frequency for Unregulated Streams in Wisconsin*[2] which considers the flooding potential 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 relate flood discharges to physical basin characteristics, namely, drainage area, wetland area, forest cover, herbaceous upland area, open water, precipitation intensity index, and soil hydraulic conductivity. 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 Project Site at Streamgage

An attachment to reference (2) above includes flood frequency discharges for 299 gaged sites computed using flood records through water year 2020. These flood frequency discharge estimates were generated using the Log-Pearson Type III (LP3) distribution method as described in Bulletin 17C 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]. Additional years of data are available from the USGS for some gaged watersheds. Flood frequency discharge estimates for these watersheds can be updated beyond water year 2020 using the same methodology as described above.

In addition to the LP3 method, reference (2) describes a theoretically improved estimate of flood discharge that combines the LP3 discharge estimate with the regression estimate for the gaged site. More details on this method can be found under the section titled "Estimating the Weighted Flood Discharge at a Streamgage."

8.2.3 Project Site at Ungaged Location on a Gaged Stream

If a project site is located on a stream with an existing streamgage (but is not at the gage itself), results obtained from the above regression equations can be combined with the flood discharge estimate at the gage to produce an improved peak flow estimate. More details are provided in the section titled ”Estimating the Flood Discharge at an Ungaged Location on a Gaged Stream” in reference (2) as discussed above. This method is applicable only if the drainage area associated with the ungaged location is between 50 and 150 percent of the drainage area associated with the streamgage.

8.2.4 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:

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

8.2.5 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. Velocities with potential to compromise slope or streambed stability are not acceptable and should be avoided. This threshold will vary depending on site geometry and local stream geomorphology.

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 are 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 8.1.3.2) 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 scour potential at a 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 and shear stress through the bridge may be the greatest. This is known as the “incipient overtopping” event. Scour should be computed for the incipient overtopping flow event if the magnitude of flow is less than the scour design flow (typically this is the 100 year flood or greater).

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 9-55 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. Public domain computer software programs most prevalent and preferred in Wisconsin bridge design work are “HEC-RAS” and “HY8”.

The HEC-RAS program is currently the most widely used methodology for floodplain and bridge hydraulic modeling. HEC-RAS should be used where existing HEC-2 data is available from a previous Flood Insurance Study. “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. 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.

The designer shall determine whether the proposed site is located in a FEMA Special Flood Hazard Area (Zone AE, A, etc). If so, a determination shall be made whether an effective hydraulic model (HEC-RAS, HEC-2, WSPRO, etc) exists for the waterway. If an effective model exists, it shall be used to evaluate the impact of the proposed stream crossing structure on mapped floodplain elevations. Areas mapped as Zone AE should always have an effective model. Effective models can be acquired from the DNR or the FEMA Engineering Library. Contact a DNR regional floodplain engineer with any questions related to existing effective models.

The designer should verify that the results of the existing hydraulic model match the flood profile listed in the corresponding Flood Insurance Study (FIS) report. This is called the ‘duplicate effective’ model. The duplicate effective model should then be updated to include geometry based on any recent project survey information. This is called the ‘corrected effective’ model and will serve as the existing condition for the bridge hydraulic analysis.

The Project Engineer shall ensure the appropriate local zoning authority is notified of the results of the hydraulic analysis.

Official bridge hydraulic models and supporting documentation are available for download from the Highway Structures Information System (HSIS).

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. HEC-RAS relies 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 ratio 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.

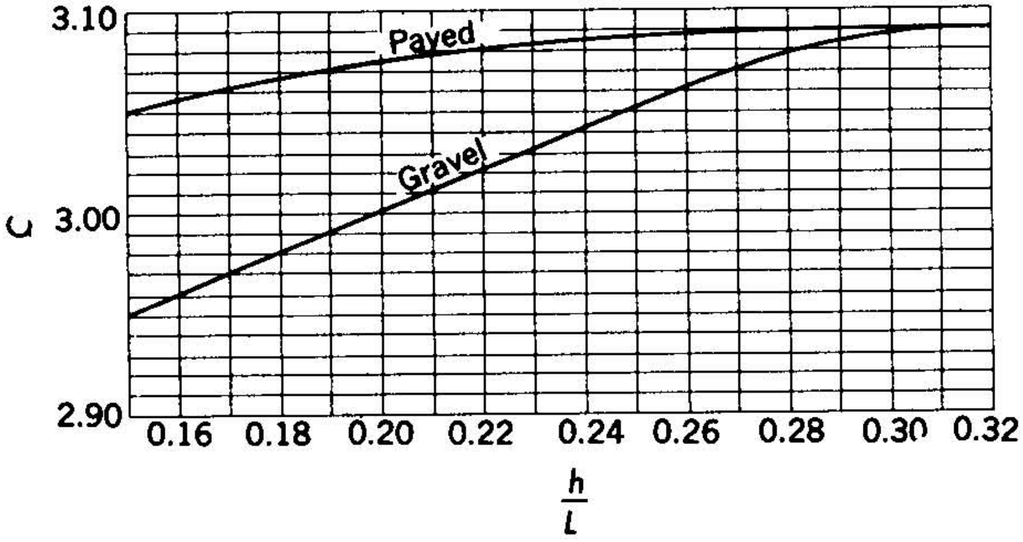

Figure 8.3-1

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

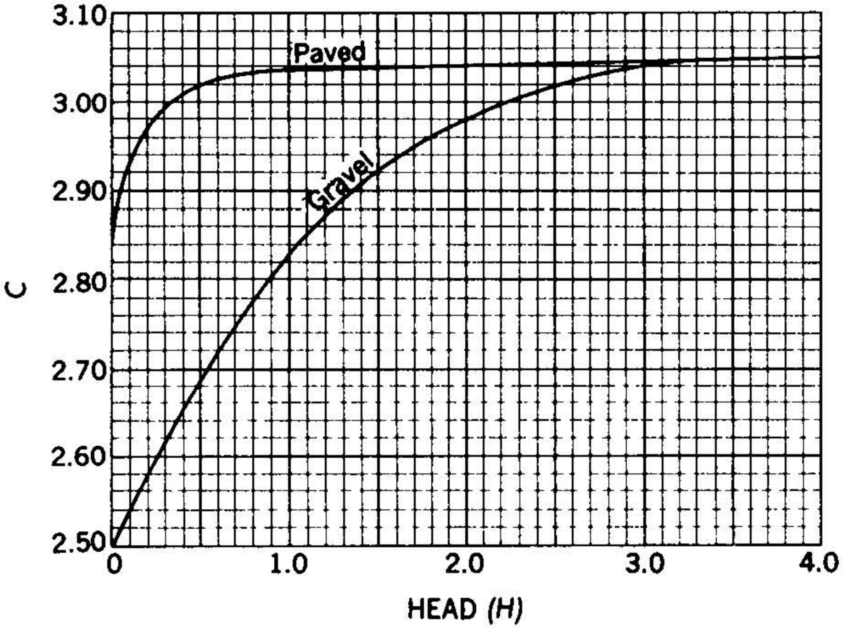

Figure 8.3-2

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

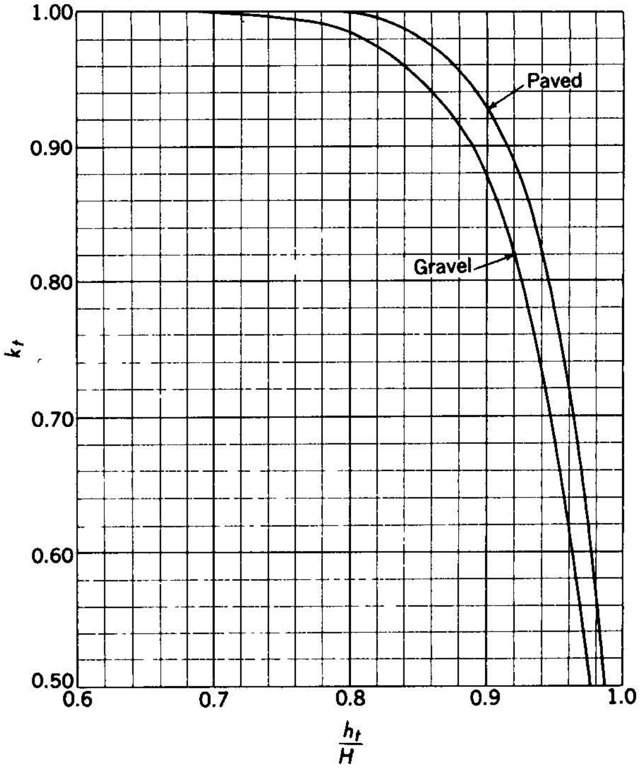

Figure 8.3-3

Definition of Adjustment Factor, k_t, for Submerged Highway Embankments

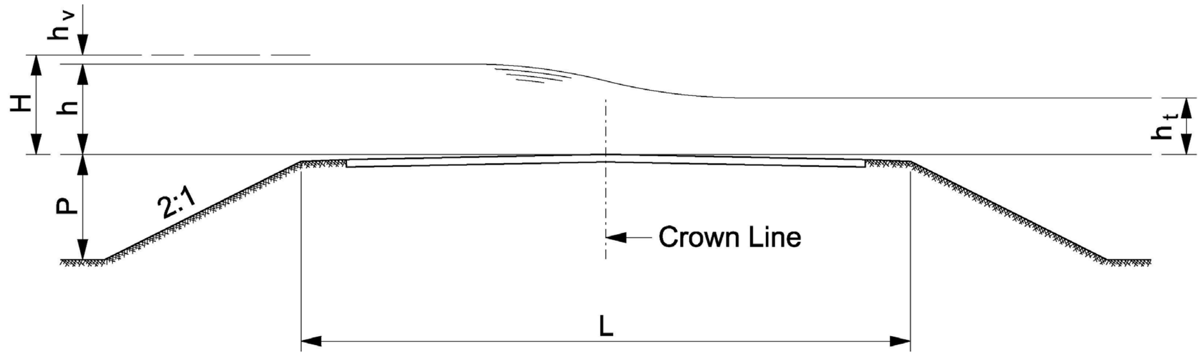

Figure 8.3-4
Definition Sketch of Flow Over Highway Embankment

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, Fifth Edition*, April 2012[14], and *Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures, Fourth Edition*, April 2012[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.

Hydraulic variables needed to complete a bridge scour analysis should be extracted from a 1D or a 2D hydraulic model. Water surface profile modeling tables highlighting pertinent variables should be included in documentation of the analysis. Scour calculations performed using the HEC RAS built-in calculators are unacceptable.

A common program used to perform a full bridge scour analysis is FHWA's Hydraulic Toolbox. Hydraulic Toolbox software and supporting documentation can be downloaded directly from FHWA's website. The hydraulic sizing report should include a discussion of scour analysis results and provide justification for scour critical code selection. FHWA's Hydraulic Toolbox can be found here: https://www.fhwa.dot.gov/engineering/hydraulics/software/toolbox404.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 placed such that the hydraulic properties of the cross section are not influenced by flow contraction at 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. Long-term Degradation (LTD) 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. Pressure flow scour depth is the greater of either LTD + Contraction Scour (Live Bed or Clear Water) or Vertical Contraction Scour. Vertical Contraction Scour should not be added to Live Bed or Clear Water Scour. 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

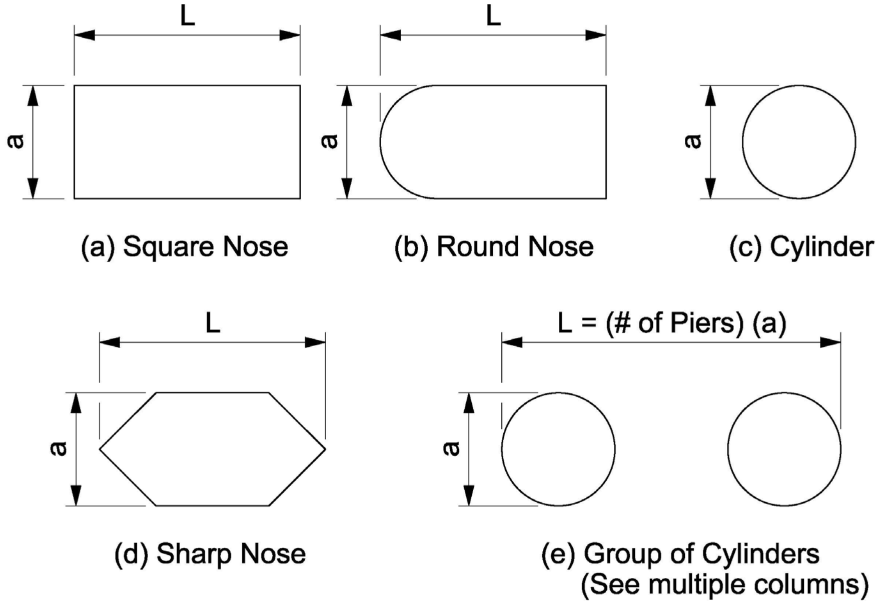

Figure 8.3-5
Common Pier Shapes

2. Abutment Scour Equations

Abutment scour analysis is dependent on equations that relate the degree of projection of encroachment (embankment) into the flood plain.

FHWA publication HEC-18 “Evaluating Scour at Bridges” strongly recommends using the NCHRP Project 24-20 methodology to assess abutment scour. 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.

Common hydraulic modeling programs used for bridge design typically provide the required hydraulic parameters needed to calculate abutment scour. Designers are cautioned to closely examine how the parameters that are used in these automated routines are defined. FHWA’s Hydraulic Toolbox software is commonly used to calculate abutment scour using the NCHRP 24-20 methodology.

The other two methods presented in HEC-18 are the Froehlich and HIRE equations. These methods 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 the NCHRP 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.

Standard slope protection at stream crossings as detailed in WisDOT Bridge Manual (Chapter 15) is not considered a designed scour countermeasure and should not be factored into scour calculations. In general, new bridges should have foundations designed to withstand calculated scour without the need for designed scour countermeasures.

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.

Assume:

$y_1 = 2.2$ and $\dfrac{V_1^2}{2 \cdot g} = 9.775 - 2.2 = 7.575'$

$V_1 = (2 \times 32.2 \times 7.575)^{1/2} = 22.1$ fps

$Q = 600 = AV = 2.2 \times \text{width} \times 22.1$, width = 12.36

$$\text{Length of flare} = \frac{\dfrac{(12.36 - 7)}{2}}{\tan 9^0} = 17'$$

$Y_1 = 2.20$

$V_1 = 22.1$

$$F_1 = \frac{V_1}{\sqrt{g \cdot y_1}} = \frac{22.1}{\sqrt{32.2 \times 2.2}} = 2.63$$

$$y_2 = y_1 \cdot \frac{1}{2} \cdot \left(\sqrt{1 + 8 \times 2.63^2} - 1\right) = 7.15$$

$L = 6(y_2 - y_1) = 6\,(7.15 - 2.20) = 29.7'$ 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.

2. Levin, S.B., and Sanocki, C.A., 2023, Estimating Flood Magnitude and Frequency for Unregulated Streams in Wisconsin: U.S. Geological Survey Scientific Investigations Report 2022–5118, 25 p., https://doi.org/10.3133/sir20225118. [Supersedes Scientific Investigations Report 2016–5140.] This report can be found on the USGS web site using the following link:

   https://pubs.usgs.gov/publication/sir20225118

3. U. S. Geological Survey, *Guidelines for Determining Flood Flow Frequency, Bulletin #17C* Revised May 2019.

4. U.S. Department of Agriculture, Soil Conservation Service, Urban Hydrology for Small Watersheds, Technical Release 55 (2nd Edition), June 1986.

5. Ven Te Chow, Ph.D. *Open Channel Hydraulics* (New York, McGraw-Hill Book Company 1959).

6. U.S. Department of Transportation, Federal Highway Administration, *Design Charts for Open-Channel Flow Hydraulic Design,* Series No. 3, August 1961.

7. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Users Manual*, (CPD-68), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

8. U.S. Army Corps of Engineers, *HEC-RAS River Analysis System Hydraulic Reference Manual* (CPD-69), Hydrologic Engineering Center, Davis CA, Version 4.0, March 2008.

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.

10. U.S. Department of Interior, Geological Survey, *Measurement of Peak Discharge at Width Contractions by Indirect Methods; Techniques of Water-Resources Investigation of the U.S.G.S.*, Chapter A4, Book 3, Third printing 1976.

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.

12. J.O. Shearman, W. H. Hirby, V.R. Schneider, H.N. Flippo, *Bridge Waterways Analysis Model*, Research Report, FHWA Report No. FHWO-RD-86/108.

13. U.S. Department of Transportation, FHWA, Hydraulic Design Series (HDS), Number 5, *Hydraulic Design of Highway Culverts*, September 2001, Revised May 2005.

14. U.S Department of Transportation, Federal Highway Administration, *Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges*, 5th Edition, April 2012.

- Provide complete description of proposed structure including calculated hydraulic characteristics.

- Provide a discussion of calculated scour depths for each scour component (LTD, Contraction, Local), total scour elevation and assigned scour code. This section should also include a discussion of proposed foundation type and depths, soil stratigraphy and ultimately a confirmation of structural stability at the total scour condition.

  Scour calculations shall be submitted with the hydraulic site report and should consist of hydraulic modeling outputs highlighting pertinent variables used for the analysis as well as output from a scour computation program (Hydraulic Toolbox, spreadsheet, etc). Scour calculations automatically performed by HEC RAS will not be accepted.

- 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, Fifth Edition	2012	FHWA-HIF-12-003	
HEC 20	Stream Stability at Highway Structures Fourth Edition	2012	FHWA-NIF-12-004	
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 Third Edition, Volume 1	2009	FHWA-NHI-09-111	
HEC 23	Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volume 2	2009	FHWA-NHI-09-112	
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	
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	

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.

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 penetrate a minimum of 10 feet through the original ground. 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 indicate that minimum pile penetration cannot be achieved, the preboring bid item should be included. 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 prior to meeting the required driving resistance. Refer to 11.3.1.6 for additional information on preboring.

Refer to 11.3.1.17.6 for additional information on scour considerations.

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.

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 Facilities Development Manual 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. It should be noted that preboring should only be used when appropriate, since many bridge contractors do not own the required construction equipment necessary for this work. Preboring is required for displacement piles when driven into new embankment with fill depths over 10 feet. For problem soils, contact the Bureau of Technical Services, Geotechnical Engineering Unit to discuss preboring considerations.

The following cases may warrant preboring:

- Displacement piles encountering a strong upper stratum with weak underlying soils. If soils (or consistent soil layers) that exhibit SPT refusal (e.g., 50 blows over 6 inches or less) are encountered prior to the scheduled pile tip elevation, pre-boring may be warranted to reduce the risk of unacceptably short pile lengths. Drivability analyses should consider harder than expected intermediate soil layers and be used to determine if preboring is warranted.

- Conditions involving short pile lengths, as discussed in 11.3.1.1. If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. For short pile length conditions, piling should be prebored at least 3 feet into solid rock and “firmly seated” on rock after placement in prebored holes. The annular space between the cored rock holes and piling should then be filled with concrete.

Other preboring considerations:

- For displacement 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. Piles placed in prebored holes founded on rock are typically firmly seated to promote firm contact between pile and rock and do not require driving or restrike to reduce the risk of pile damage.

- The annular space between the prebored hole and piling is required to be backfilled. After the pile is installed, concrete should be used to the top of the rock to properly socket point resistance piles. Clean sand should then be used to backfill the remaining annular space. Backfill material should be deposited with either a tremie pipe or concrete pump to reduce potential arching (bridging) and assure that the complete annular space is filled. Backfill materials for prebored holes should be clearly indicated in the plan documents.

- Some sites may require casing during the preboring operation. If casing is required, it should be clearly indicated in the plan documents.

See 11.3.1.17.6 for scour considerations.

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

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. During design, estimated pile lengths may require an increase 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)

The Site Investigation Report shall determine if preboring is necessary. Additionally, Special Provisions and/or plan notes may also be necessary to address unique preboring requirements. This may include, but is not limited to indicating minimum pile embedments, minimum pile tip elevations, and clarifying payment for preboring.

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 considered 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.

						Modified Gates Driving Criteria		PDA/CAPWAP Driving Criteria	
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)	Factored Resistance (Pr) ($\phi = 0.50$) (tons)	Required Driving Resistance (Rn_{dyn}) (tons) (5)	Factored Resistance (Pr) ($\phi = 0.65$) (tons)	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 (11)	72 (8)	110 (11)
10 ¾“	0.250	82.5	98.2	0.75	74	65 (8)	130 (11)	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 (11)	104 (8)	160 (11)
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 (11)	111 (8)	170 (11)
14”	0.375	137.9	164.1	0.75	123	120 (8)	240 (11)	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 (11)	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 [Eq’n 5.6.4.4-3]**. $k_C = 0.85$ (for $f'_C \leq 10.0$ ksi). Neglecting the steel shell, equation reduces to $0.68 * f'c * Ag$.

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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 and structural design of abutment supports shall be in accordance *AASHTO LRFD*. No additional guidance is available at this time.

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. See 12.11 for bridge approach design considerations and Chapter 11-Foundation Support for additional information on deep foundations.

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 allowable slopes at the abutment. A slope of 2:1 is usually used, and a slope greater than 2:1 is usually not permitted. Current practice is to round up to the next available wing length based on 2 feet increments and to consider an additional 2 feet to match other wing lengths. 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. Roadway embankment slopes are typically limited to a slope of 2.5:1 and may require a traffic barrier. Refer to the Facilities Development Manual (FDM) for roadway embankment slopes and traffic barrier requirements.

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. Wing lengths should be lengthened an additional 2 feet to allow for the finished grade to intersect the top of wall 2 feet from the end of wings for erosion control protection. The additional 2 feet of wing wall length is only intended for wings parallel to the roadway.

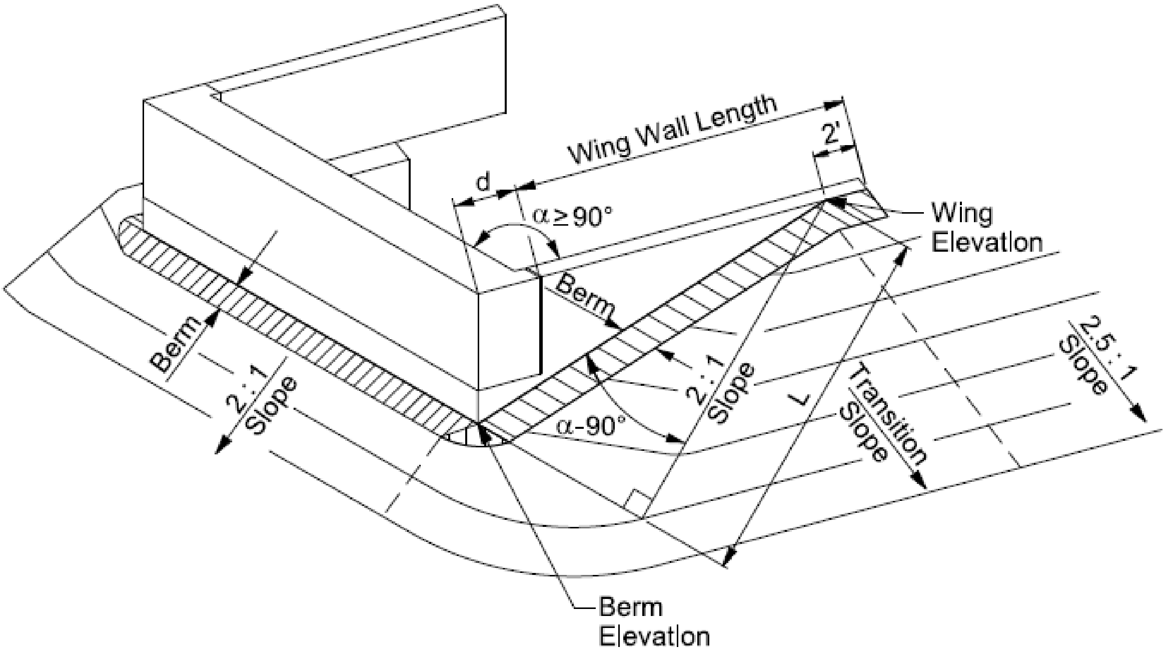


Figure 12.4-1

Wings Parallel to Roadway and Wing Wall Angle ≥ 90°

For wing wall angle, $\alpha \geq 90°$:

$$L = (\text{Wing Elevation} - \text{Berm Elevation})\ (2)$$

$$\text{Wing Wall Length} = \frac{L}{\cos(\alpha - 90^\circ)} - d + 2.0 \text{ feet}$$

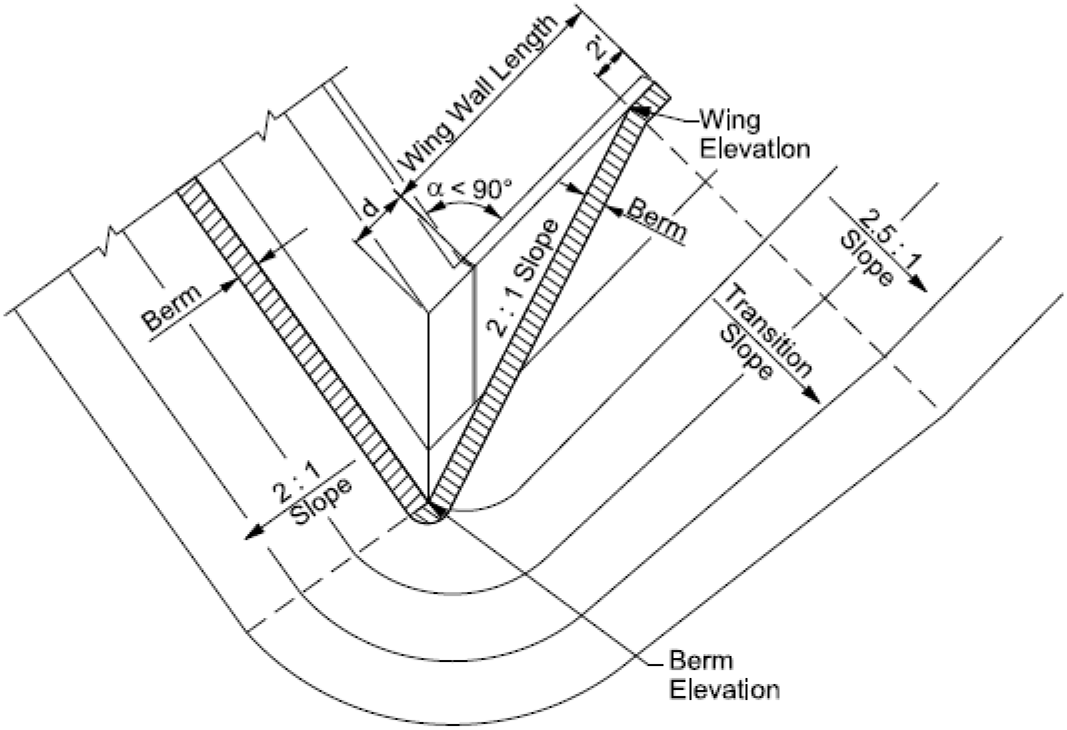


Figure 12.4-2

Wings Parallel to Roadway and Wing Wall Angle < 90°

For wing wall angle, $\alpha < 90°$:

Wing Wall Length = (Wing Elevation – Berm Elevation) (2) – d + 2.0 feet

Note: The above calculations provide the minimum required wing wall length and should be rounded accordingly.

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.

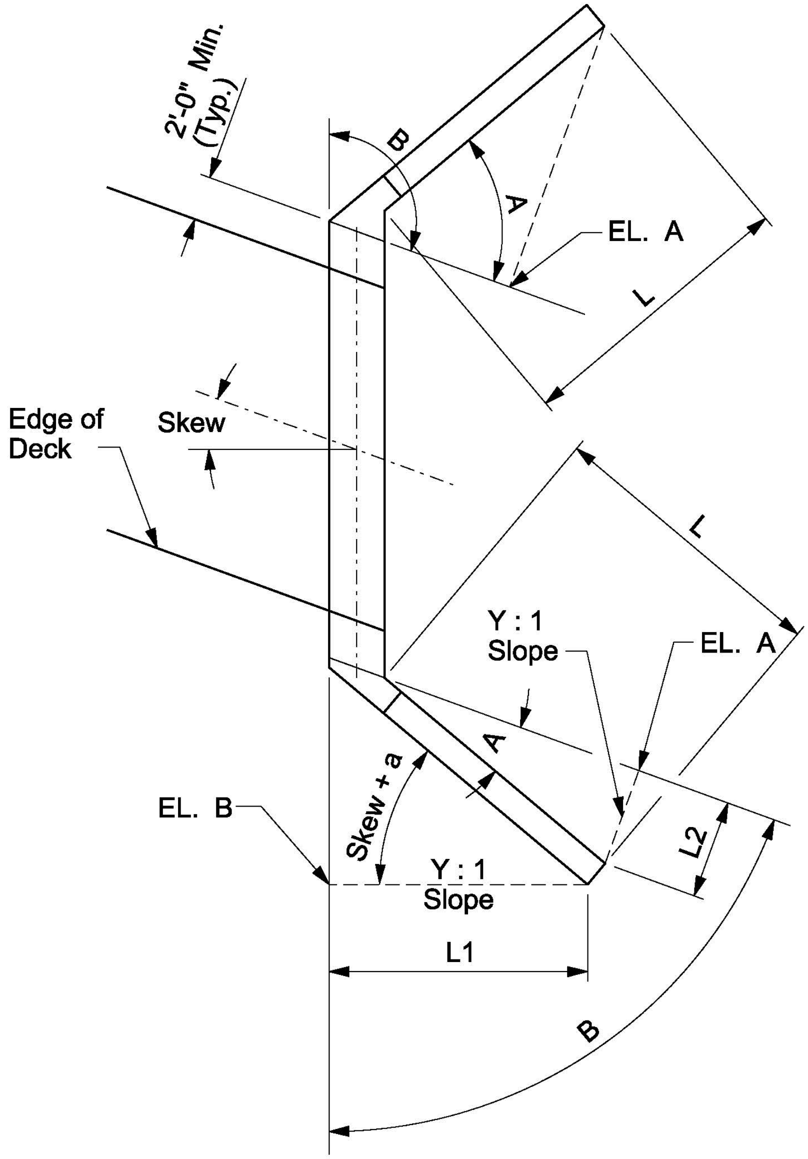


Figure 12.4-3
Wings Not Parallel to Roadway and Equal Slopes

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. Provide a minimum slope of 0.5% and discharge to suitable drainage (i.e. a storm sewer system or ditch). It is best to place the pipe underdrain along 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. For bottom of abutments located below the normal water, pipe underdrain should be sloped to discharge a minimum of 1 foot above the normal water elevation.

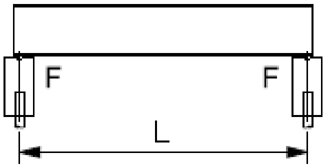
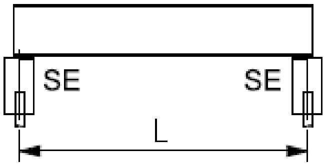
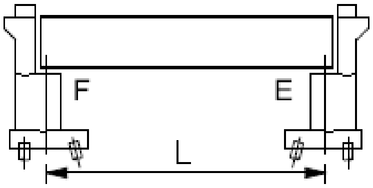
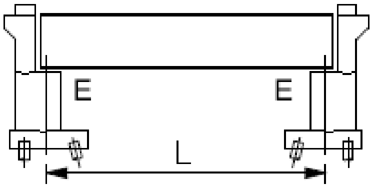
Pipe underdrains and weepholes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks.

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	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A1 (F-F) 	a. L ≤ 150' S ≤ 30° AL ≤ 50'	a. L ≤ 150' S ≤ 15° AL ≤ 50' 28" only (36W" thru 82W" require SE)	a. L ≤ 150' S ≤ 15° AL ≤ 50'
Type A1 (SE-SE) 	a. L ≤ 300' S ≤ 30° AL > 50'	a. L ≤ 300' S ≤ 40°	a. L ≤ 150' S ≤ 40°
Type A3 (F-E) 	Not used	Single span and (S > 40°)	Single span and (L > 150' or S > 40°)
Type A3 (E-E) 	b. L >300' and S ≤ 30° with rigid piers	L>300' or (S > 40° and multi-span)	Multi-span and (L>150' or S > 40°) with rigid piers

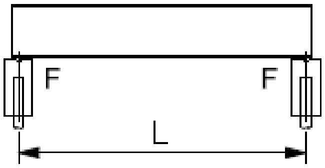
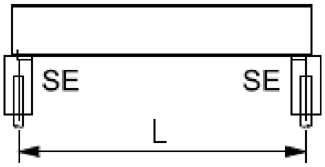
Abutment Arrangements	Superstructures		
	Concrete Slab Spans	Prestressed Girders	Steel Girders
Type A5 (F-F) 	L ≤ 150' S ≤ 30° AL ≤ 50'	L ≤ 150' S ≤ 15° AL ≤ 50' 28" only (36W" thru 82W" require SE)	L ≤ 150' S ≤ 15° AL ≤ 50'
Type A5 (SE-SE) 	L ≤ 200' S ≤ 30° AL > 50'	L ≤ 200' S ≤ 30°	L ≤ 150' S ≤ 30°
ABUTMENT TYPES			

Figure 12.7-1
Recommended Guide for Abutment Type Selection

Where:

S = Skew

AL = Abutment Length

F = Fixed seat

SE = Semi-Expansion seat

E = Expansion seat

L = Length of continuous superstructure between abutments

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.

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 14-10-25) 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:

$$R_{DC} = (1.10\ \mathrm{K/ft})\left(\frac{60\ \mathrm{Feet}}{2}\right) = 33.0\ \mathrm{kips}$$

$$R_{DW} = (0.18\ \mathrm{K/ft})\left(\frac{60\ \mathrm{Feet}}{2}\right) = 5.4\ \mathrm{kips}$$

These dead loads are illustrated in Figure 12.8-1. The dead loads are equally distributed over the full length of the abutment.

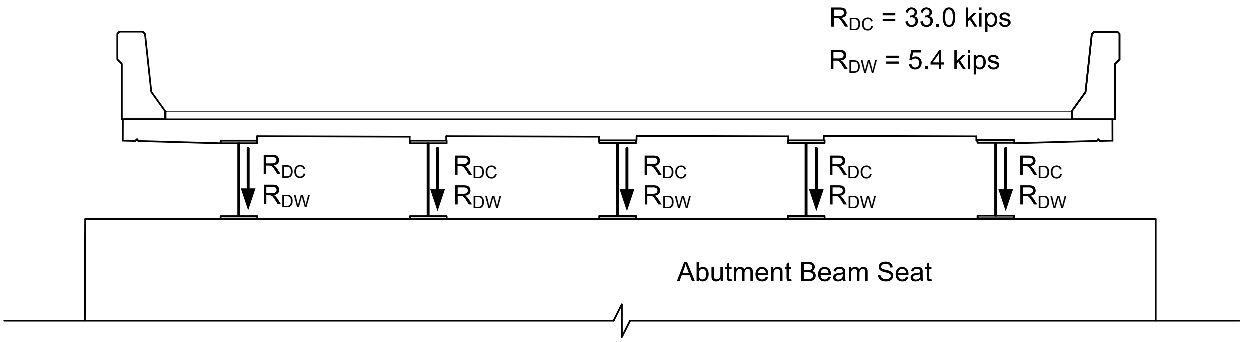


Figure 12.8-1
Dead Load on Abutment Beam Seat

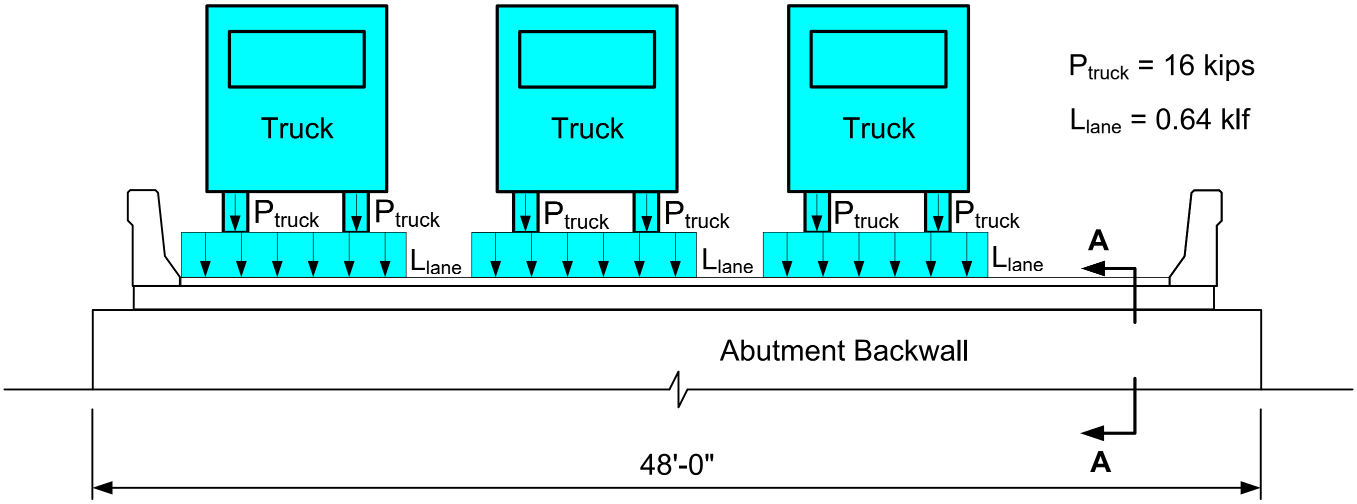
The next step is to compute the live load applied to the abutment. To compute live load reactions to bearings, live load distribution factors must be used to compute the maximum live load reaction experienced by each individual girder. However, to compute live loading on abutments, the maximum number 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.

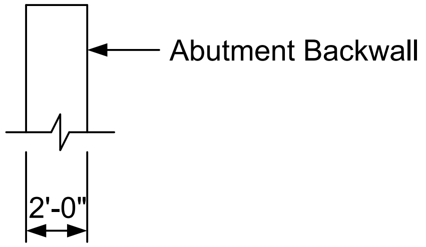
To illustrate the computation of live loads for abutment design, consider the same 60-foot simple span bridge described previously. Since the roadway width is 44 feet, the maximum number of design lanes is three (44 / 12 = 3.67 ≈ 3 lanes). The backwall live load is computed by placing the three design 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:

$$R_{\text{LL backwall}} = \frac{(0.85)\left[(3\text{ lanes})\left(\frac{2\text{ wheels}}{\text{lane}}\right)\left(\frac{16\text{ kips}}{\text{wheel}}\right)(1.33) + (3\text{ lanes})(0.64\text{ klf})(2.0\text{ feet})\right]}{48\text{ feet}}$$

$$= 2.33\,\frac{\text{K}}{\text{ft}}$$

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.





Section A-A

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:

$$R_{\text{LL stem}} = \frac{(3\ \text{lanes})(0.85)\left[(60.8\ \text{kips})(1.33)+(19.2\ \text{kips})\right]}{48\ \text{feet}} = 5.32\ \frac{\text{K}}{\text{ft}}$$

This live load configuration for an abutment beam seat is illustrated in Figure 12.8-3.

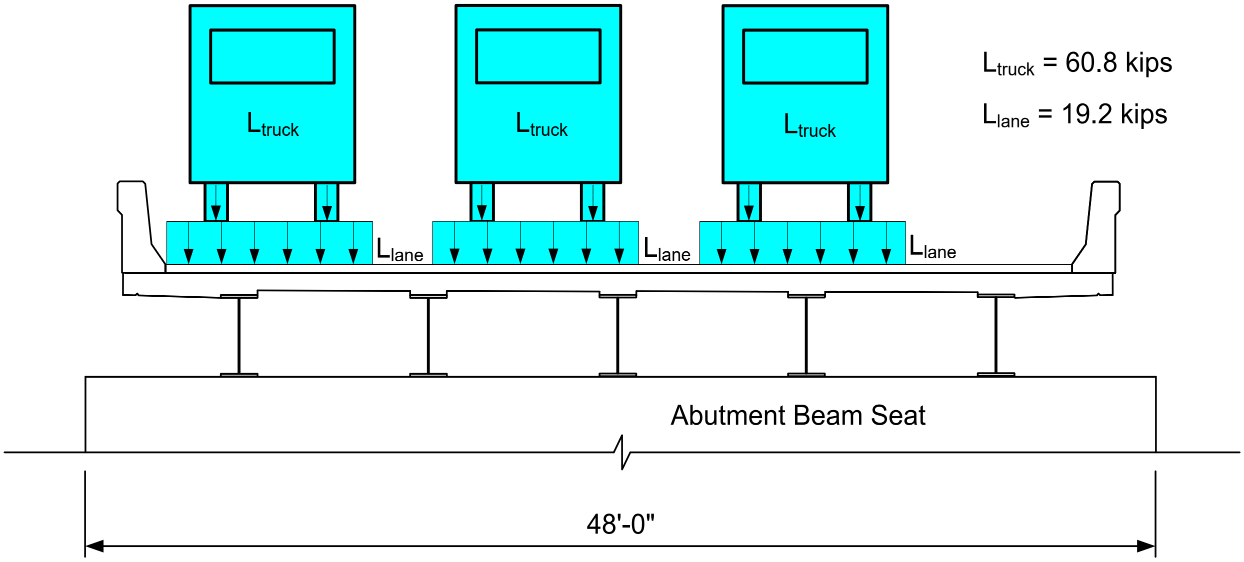


Figure 12.8-3
Live Load on Abutment Beam Seat

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:

$$R_{LL\ footing} = \frac{(3\ \text{lanes})(0.85)\left[60.8\ \text{kips} + 19.2\ \text{kips}\right]}{48\ \text{feet}} = 4.25\,\frac{K}{ft}$$

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 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 geotechnical engineer should evaluate approaches for settlement susceptibility and provide recommendations for mitigating settlements prior to approach placement. 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. Structural approach slabs are not intended to mitigate excessive approach settlements.

WisDOT policy item:

Structural approach slabs shall be used on all Interstate and US highway bridges. Structural approach slabs are recommended for bridges carrying traffic volumes greater than 3500 AADT in the future design year. Structural approach slabs are not required on buried structures and culverts. Structural approach slabs should not be used on rehabilitation projects, unless approved otherwise. Other locations can be considered with the approval of the Chief Structural Design Engineer. Design exceptions to structural approach slabs are considered on a project-by-project basis.

Standards for Structural Approach Slab for Type A1 and A3 Abutments and Structural Approach Slab Details for Type A1 and A3 Abutments are available for guidance.

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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 are 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 an initial protective deck seal. This includes all deck, sidewalk, median, paving notch, and concrete overlay surfaces. For decks with open rails, the deck seal shall wrap around the edge of deck and include 1’-0” underneath the deck. A pigmented seal shall be used on the top and inside faces of parapets. After the initial deck seal, decks shall be resealed at regular intervals or receive a thin polymer overlay as described in Chapter 40 – Bridge Rehabilitation. Refer to the Standard drawing in Chapter 17 – Superstructure-General for additional information.

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. HPC structures with a design speed of 40 mph or greater shall use bid item “Longitudinal Grooving”, unless directed otherwise. Longitudinal grooving improves the curing process, reduces tire noise, and restores friction. Groove surfaces prior to opening the bridge to traffic. If a polymer overlay will be placed on an HPC structure prior to opening to traffic, then longitudinal grooving can be eliminated.

- Polymer overlays - This system extends the decks service life before rehabilitation is required. Refer to Chapter 40 for additional information.

- 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.

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The designer shall investigate all welded connections to a tension flange. Calculate and show the tension zones on top and bottom flanges for all continuous steel girders on the contract plans. The defined tension zone will assist with inspection and prohibit field welding within the tension zone, unless noted otherwise (i.e. shear connectors). Field welding within the tension zone for construction purposes (i.e. deck form attachments) is prohibited. See Chapter 6-Plan Preparation for additional guidance.

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:

$$f_b = \frac{\text{DLM (DC1)}}{\text{S(steel only)}} + \frac{\text{DLM (DC2 \& DW)}}{\text{S(steel only)}} + \frac{\text{LLM (Traffic)}}{\text{S(steel only)}} + \frac{\text{LLM (Pedestrian)}}{\text{S(steel only)}}$$

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:

$$f_b = \frac{\text{DLM (DC1)}}{\text{S(steel only)}} + \frac{\text{DLM (DC2 \& DW)}}{\text{S(composite,3n)}} + \frac{\text{LLM (Traffic)}}{\text{S(composite,n)}} + \frac{\text{LLM (Pedestrian)}}{\text{S(composite,n)}}$$

For composite sections, flexural stresses in the concrete deck subjected to positive flexure are computed as follows:

$$f_b = \frac{\text{DLM(DC2} + \text{DW)}}{\text{S(composite,n)}} + \frac{\text{LLM(Traffic)}}{\text{S(composite,n)}} + \frac{\text{LLM(Pedestrian)}}{\text{S(Composite,n)}}$$

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]**). 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, conforming to ASTM F3125, 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 the surfaces in contact (faying):

- For steel with fully painted surfaces, use Ks = 0.30.
- 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 [6.13.6.1.3a]**, 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 [6.13.6.1.3a,b]** and **LRFD [C6.13.6.1.3a,b]**.

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 [C6.13.6.1.3a,b]**, 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 [6.13.6.1.3a,b]**. 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 [6.13.6.1.3a,b]**, 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 [6.13.6.1.3a,b]**

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.

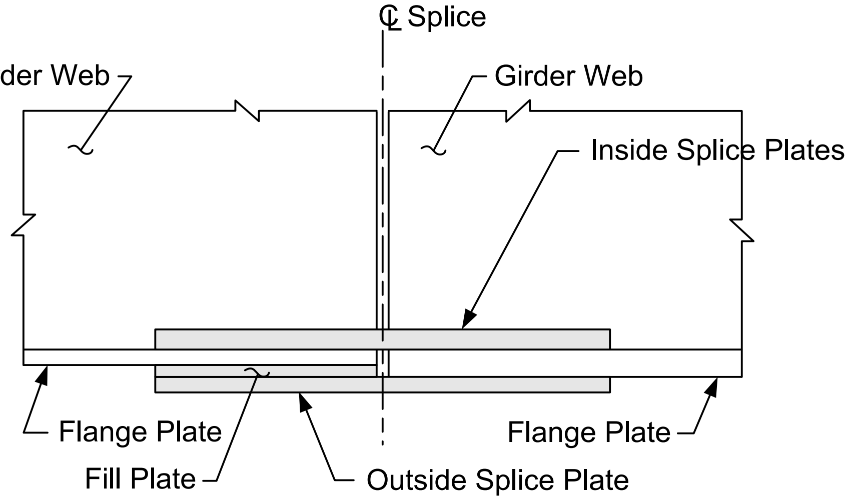


Figure 24.8-1
Bottom Flange Splice Configuration

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 [C6.13.6.1.3a,b]**. 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.

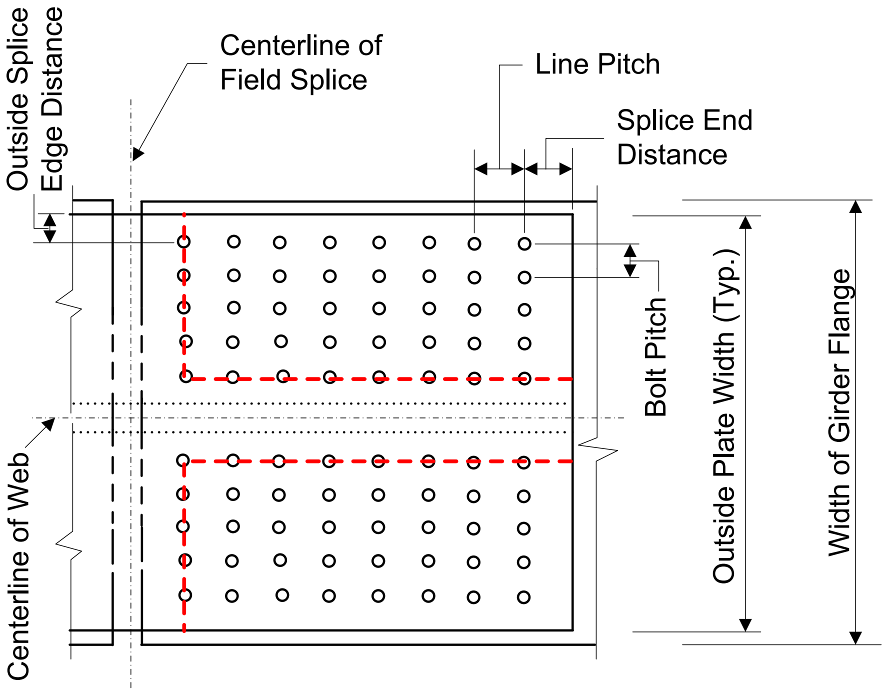


Figure 24.8-2
Double – L Block Shear Path, Flange and Splice Plates

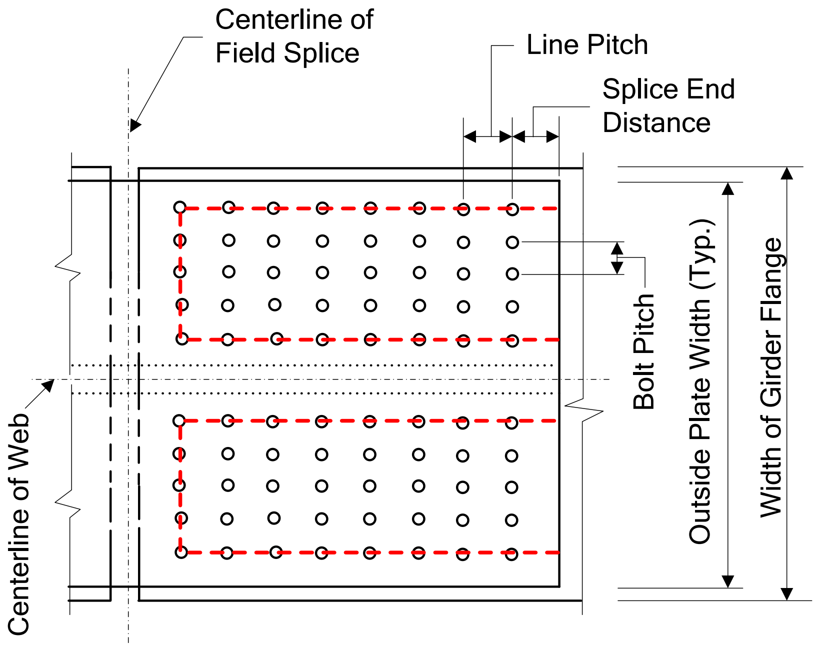


Figure 24.8-3
Double – U Block Shear Path, Flange and Splice Plates

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 [6.13.6.1.4]**. 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 [6.13.6.1.4]**.

24.8.3.4.2 Slip Resistance

As specified in **LRFD [6.13.6.1.3a,b]**, 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 smaller of the nominal shear resistance of the individual bolts and the nominal bearing resistance of the individual bolt holes parallel to the line of the applied force. Nominal shear resistance of the bolt is found in **LRFD [6.13.2.7]**.

If the bearing resistance controls and is not adequate, 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 [6.13.6.1.3a,c]**, 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 [C6.13.6.1.3c]**. 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.

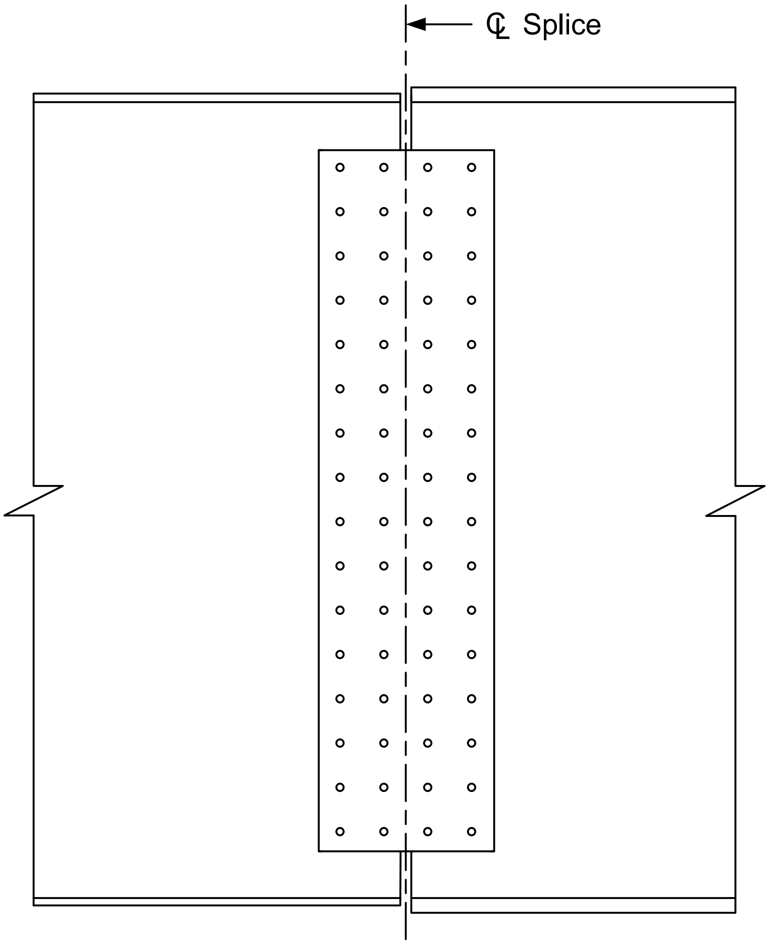


Figure 24.8-4
Web Splice Configuration

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 [6.13.6.1.3a,c]**.

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 [6.13.6.1.3a,c]**. 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.

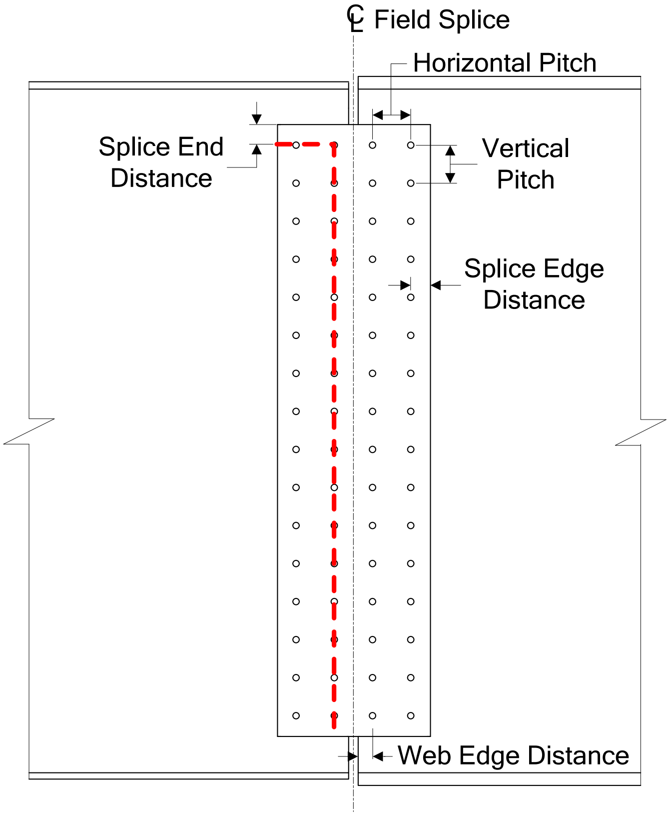


Figure 24.8-5
Block Shear Path, Web Splice

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 [6.13.6.1.3a,c]**.

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 [6.13.6.1.3a,c]**. 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.

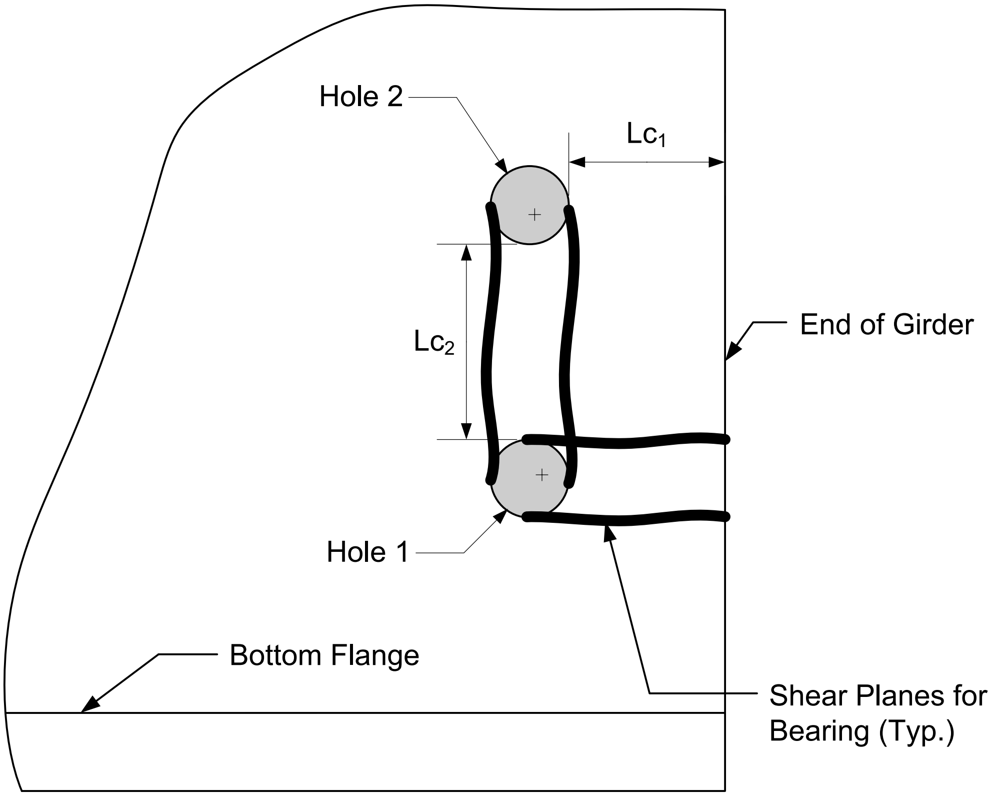


Figure 24.8-6
Bearing Resistance at Girder Web Bolt Holes

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. Calculate the bearing resistance at bolt holes using the appropriate equations in **LRFD [6.13.2.9]**. The design bearing strength of the connected material is calculated as the sum of the smaller of the nominal shear resistance of the individual bolts and the nominal bearing resistance of the individual bolt holes. Nominal shear resistance of the bolt is found in **LRFD [6.13.2.7]**. If the bearing resistance controls and is not adequate, 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.

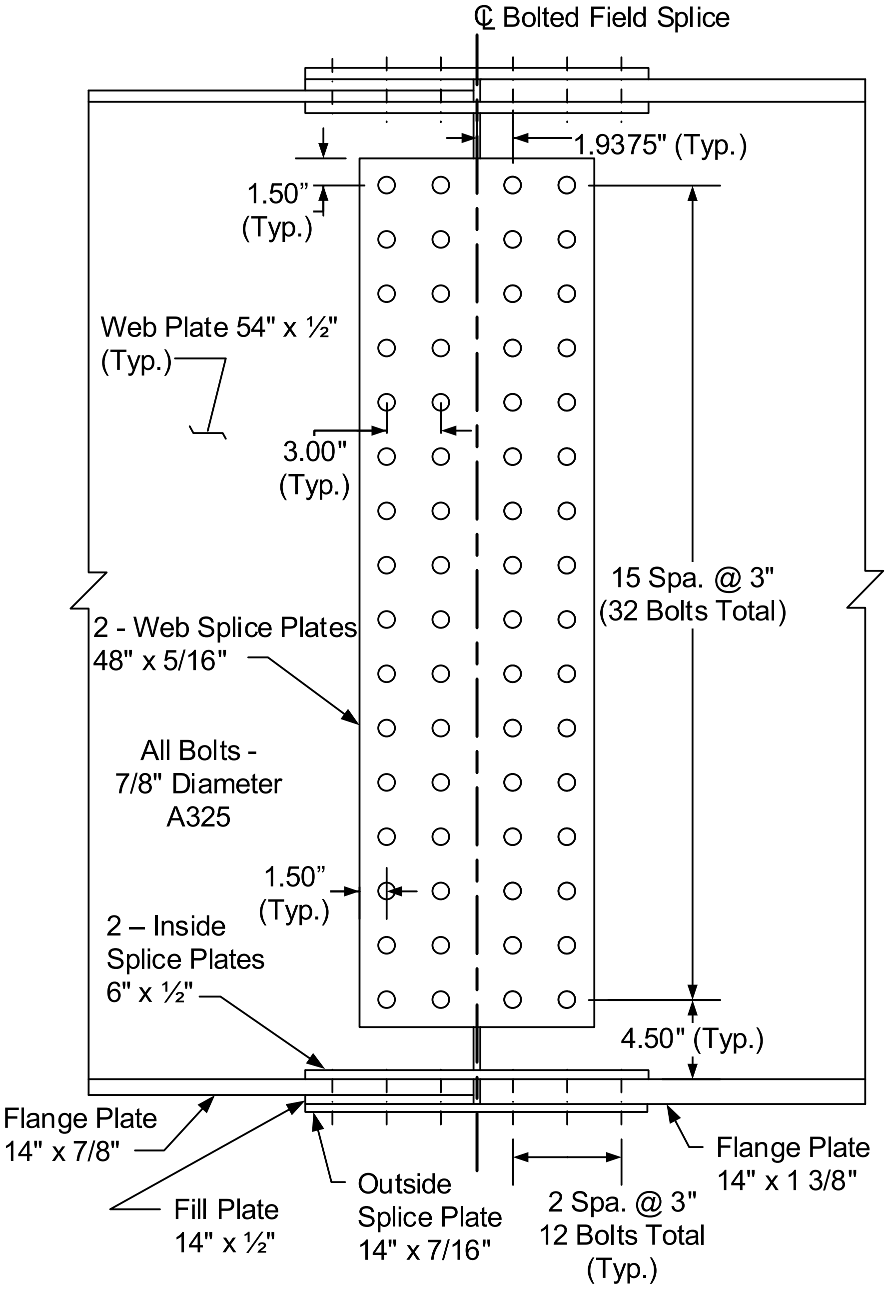


Figure 24.8-7

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.

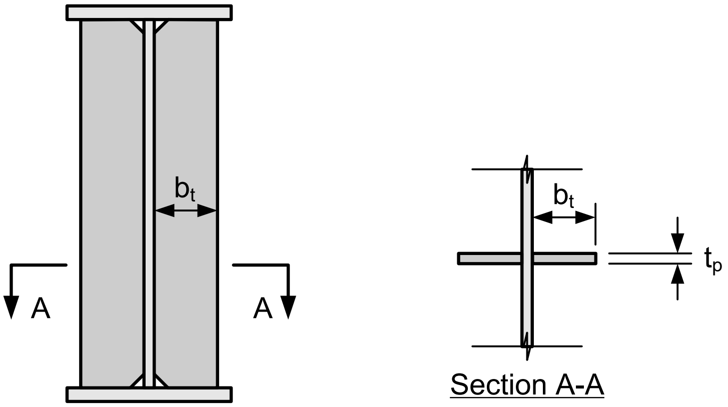


Figure 24.9-1
Projecting Width of a Bearing Stiffener

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 [Eq'n 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]** (= 0.95) - (axial compression - steel only)

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 $\lambda \leq 2.25$, then: $P_n = 0.658^{\lambda} \cdot F_{ys} \cdot A_s$

If $\lambda > 2.25$, then: $P_n = (0.877 \cdot F_{ys} \cdot A_s) / \lambda$

Where:

$$\lambda = P_o / P_e = (K\ell / r_s \cdot \pi)^2 \cdot F_{ys} / E \;;\quad P_e = \pi^2 \cdot E \cdot A_s / (K\ell / r_s)^2 \;;\quad P_o = F_{ys} \cdot A_s$$

E = Modulus of elasticity of steel (ksi)

P_o = nominal yield resistance (kip)

P_e = elastic critical buckling resistance (kip) **LRFD [6.9.4.1.2]**

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})

$K\ell$ = 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.

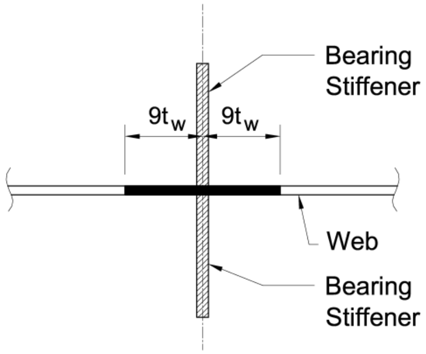


Figure 24.9-2
Effective Column Section for Welded Bearing Stiffener Design (One Pair of Stiffeners)

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. See 24.6.21 for additional information on welded connections.

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.

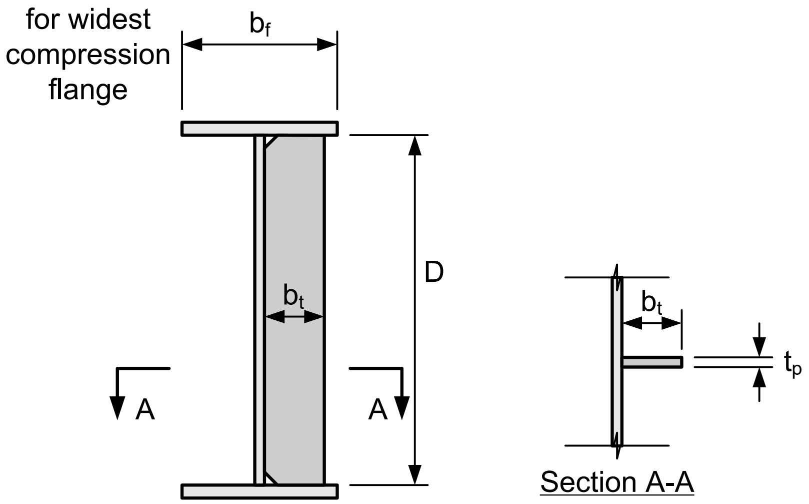


Figure 24.10-1
Projecting Width of Transverse Stiffeners

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:

$$I_t \geq \left(\frac{b_t}{b_\ell}\right)\left(\frac{D}{3d_o}\right)I_\ell$$

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_ℓ = Projecting width of the longitudinal stiffener (in.)

D = Web depth (in.)

d_o = Smaller of the adjacent web panel widths (in.)

I_ℓ = 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 constructability 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_ℓ, of the longitudinal stiffener must satisfy the following requirement in order to prevent local buckling of the stiffener plate:

$$b_\ell \leq 0.48 t_s \sqrt{\frac{E}{F_{ys}}}$$

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:

$$I_\ell \geq D t_w^3 \left[2.4 \left(\frac{d_o}{D} \right)^2 - 0.13 \right] \beta$$

Where:

I_ℓ = 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.

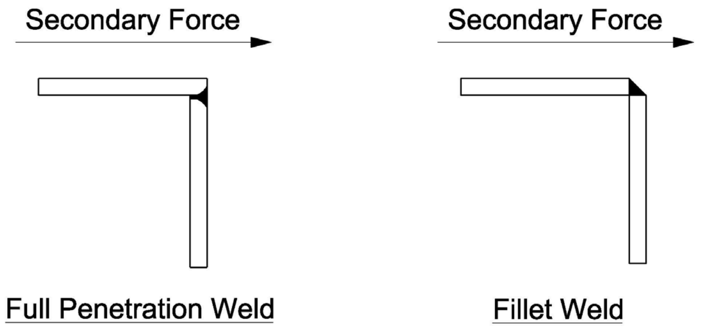


Figure 24.12-1
Welds for Built-up Box Sections

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 $R_h F_{yc}$ 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 $\phi_f F_{crw}$ 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*.

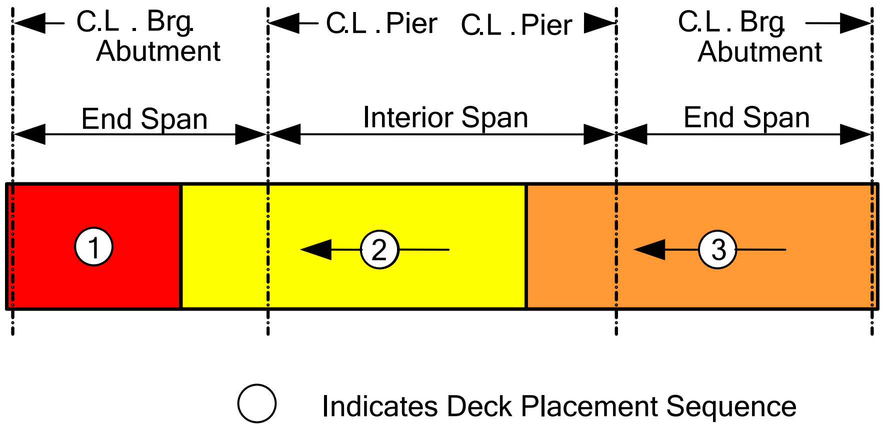


Figure 24.12-2
Deck Placement Sequence

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.

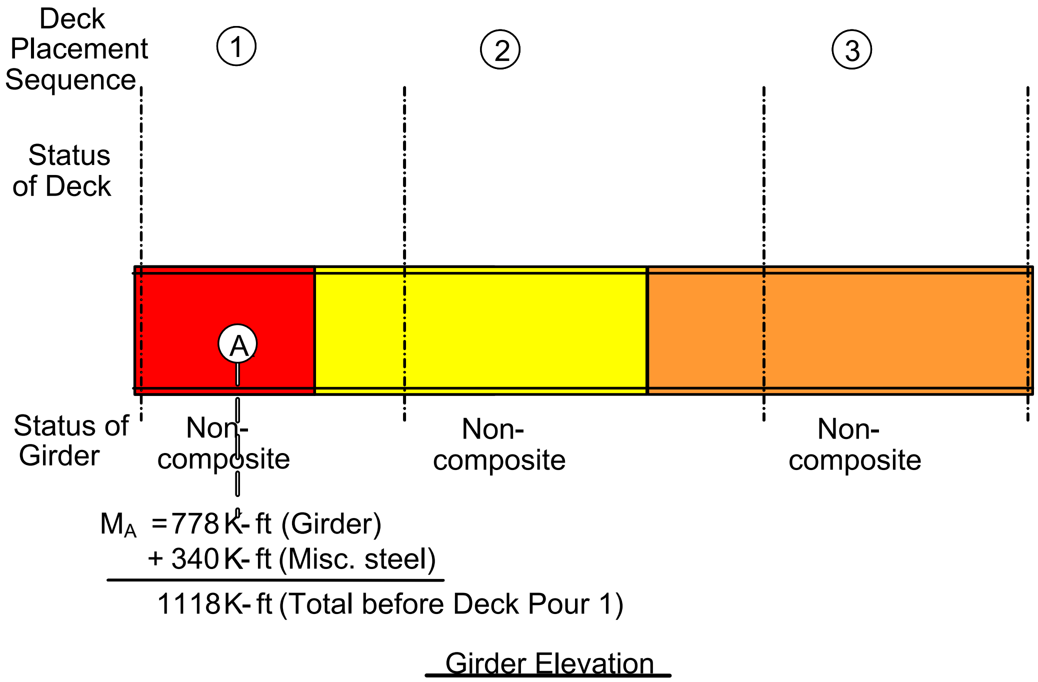


Figure 24.12-3
Girder Elevation View

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).

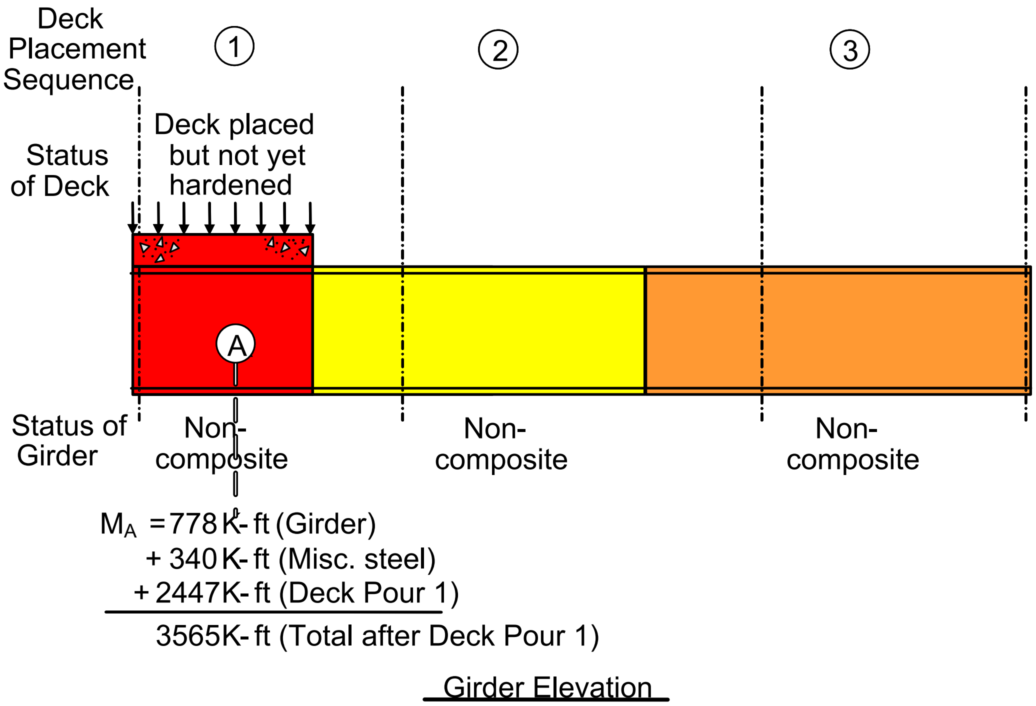
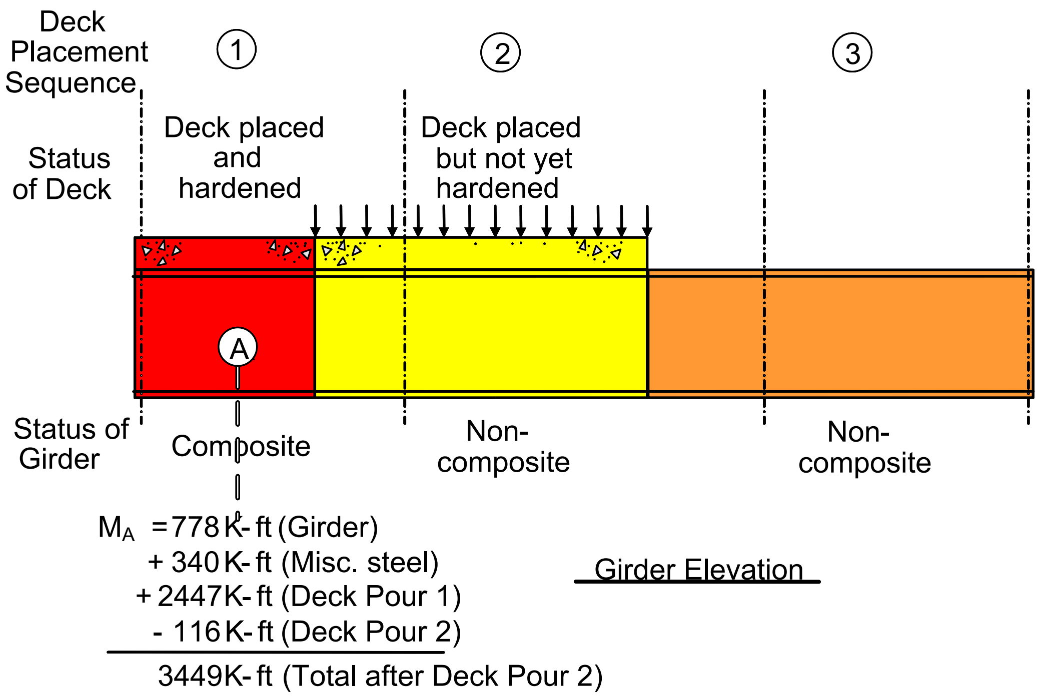


Figure 24.12-4

Deck Placement Analysis 1

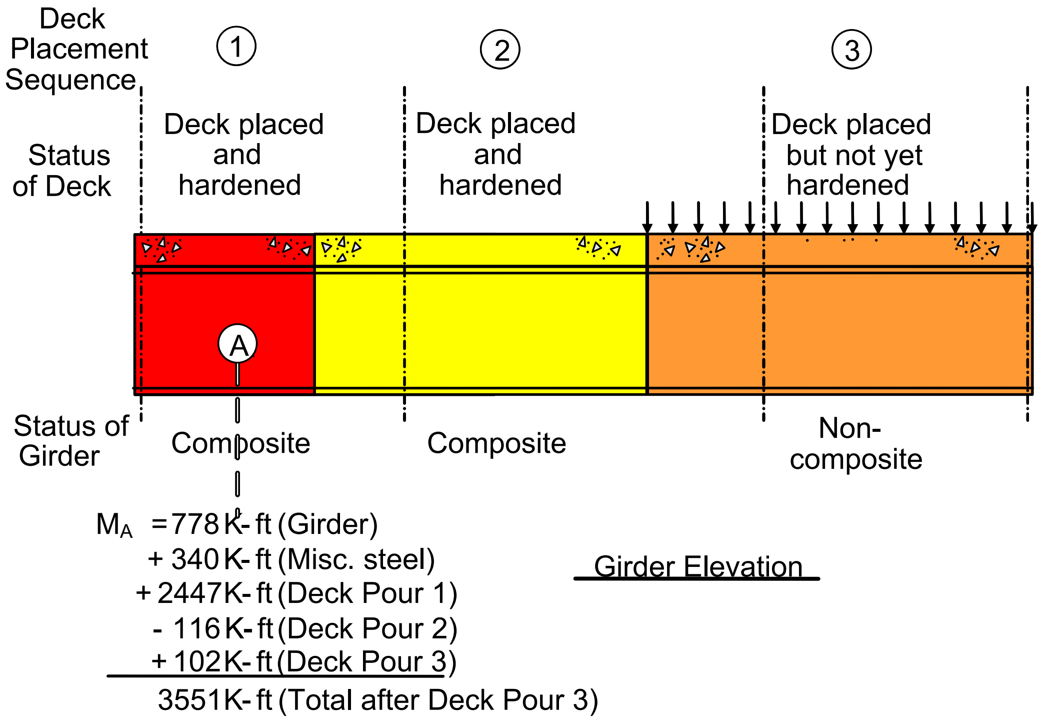
The next deck placement (Cast 2) is located immediately adjacent to Cast 1, as shown in Figure 24.12-5. The concrete in the first placement is now assumed to be hardened so that those portions of the girder are now composite. Therefore, as required in **LRFD [6.10.3.4]**, those portions of the girder 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.



M_A = 778 K-ft (Girder)
+ 340 K-ft (Misc. steel)
+ 2447 K-ft (Deck Pour 1)
- 116 K-ft (Deck Pour 2)
3449 K-ft (Total after Deck Pour 2)

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.



M_A = 778 K-ft (Girder)
+ 340 K-ft (Misc. steel)
+ 2447 K-ft (Deck Pour 1)
- 116 K-ft (Deck Pour 2)
+ 102 K-ft (Deck Pour 3)
3551 K-ft (Total after Deck Pour 3)

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 *AMS Standard Color Numbers* 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.

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:

Certain criteria must be met to consider a trapezoidal steel box girder bridge to be a System Redundant Member (SRM), as outlined in *A Simplified Approach for Designing SRMs in Composite Continuous Twin-Tub Girder Bridges* (as summarized in Appendix A – the full report is available upon request from BOS) by Robert J. Connor, et. al., Purdue University. A summary of these steps required by WisDOT are outlined below this policy item box.

It is required to design twin-tub girders to meet SRM criteria. BOS approval is required for all box girders.

Summary of Appendix A

Approach
For a multi-span twin-tub girder bridge to be considered an SRM, the bridge must meet certain screening criteria. If the criteria are met, design must be in accordance with the provisions set forth in the subject report. Figure A-1 is a flowchart for describing the proposed guideline steps.

Screening
To consider a twin-tub girder an SRM, certain criteria must be met, which require continuous spans, composite section with specific shear stud design, maximum bridge width, maximum girder spacing, web depth range, interior span length limits, exterior span length limits, ratio of unfractured to fractured span length limits, ratio of radius of curvature to longest span length limit, skew limit, maximum number of design lanes, and maximum dead load displacement limit at both interior and exterior spans.

Design Methodology
If the screening criteria are met, the design then needs to meet specific design requirements for shear studs, intermediate diaphragms, bottom flange buckling resistance, and positive moment flexural resistance.

Additional information regarding design and rating includes:

New twin steel tub girder designs should continue to include the redundancy load factor (**LRFD [1.3.4]**) for nonredundant members, η_R = 1.05 under the strength limit state, regardless of the structure’s final redundant related classification (e.g. FCM or SRM). The continued use of this load factor, even if a structure is determined to be redundant via system redundant classification is to maintain consistency in design with the original group of structures evaluated and documented in the report by Purdue University.

However, the load redundancy factor shall not be considered when checking the *Redundancy I and II* limit states described in the aforementioned report.

For load ratings, the *Manual for Bridge Evaluation*, section 6A.4.2.4 applies a system factor ϕ_S = 0.85 to the resistance of welded members in two-girder systems (i.e. twin steel tub girders). If a twin steel tub girder bridge has achieved SRM classification the system factor should be taken as 1.0 for load rating purposes.

24.16 Design Examples

E24-1	2-Span Continuous Steel Plate Girder Bridge, LRFD
E24-2	Bolted Field Splice, LRFD

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39.1.6 Full Span OSS Selection Criteria

Full Span OSS Type	Design	Span Length [1]	Vertical Support Height [1]	Static Sign Total Area & Max. Dimensions		DMS Max. Dimensions & Max. Weight [1]
Monotube	Contractor Designed	40'-0" Min. 75'-0" Max.	25'-0" Max. Column Base Plate to CL of Monotube Arm	Sign Area ≤ 150 SF Max. Sign Height ≤ 5'-0"	OR	Not Used
2-Chord Truss	Contractor Designed	40'-0" Min. 100'-0" Max. (static) / 70'-0" Max. (DMS)	27'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 300 SF Max. Sign Height ≤ 10'-0"		10'-6"W x 6'-0"H Max. 850 Lbs. Max
4-Chord Truss	Standard Design	40'-0" Min. 130'-0" Max.	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ Note 2 Max. Sign Height ≤ 12'-0" Note 3		26'-0"W x 9'-0"H 4,500 Lbs. Max.
4-Chord Truss	Non-Standard Design	>130'-0"	Column Height Exceeds Limit for Standard Design	Sign Area or Height Exceeds Limits For Standard Design		DMS Dimensions or Weight Exceeds Limits For Standard Design

Table 39.1-3
Full Span OSS Selection Criteria

Note 1: The limiting parameters of length, height and sign area are depicted in 39.6 for Contractor Designed OSS and on the Standard Design Drawings for the 4-Chord OSS.

Note 2: Maximum sign area for full span 4-chord standard design = 12' x (90% * Span Length).

Note 3: For sign panel heights greater than 12'-0" but less than or equal to 15'-0" in height, contact BOS to discuss applicability of standard designs. Sign panels greater than 15'-0" in height require non-standard designs and the aluminum I-beams that connect the panel to the truss must be designed for the additional unbraced length.

39.1.7 Butterfly and Butterfly Truss OSS

OSS Type	**Design**	**Static Sign Total Area & Max. Dimensions**[2]		**DMS Total Area & Weight**
Butterfly	Standard Design	Sign Area ≤ 200 Sq. Ft. Sign Height ≤ 10’-0”	OR	N.A.
Butterfly Truss [1]	Non-Standard Design	Sign area > 200 sq. ft. Sign Height > 10’-0”		See 4-Chord full span requirements. Limit 2 per structure.

Table 39.1-4
Butterfly and Butterfly Truss OSS Selection Criteria

Note 1: Butterfly Trusses should use the WisDOT 4-chord cantilever truss dimensions (3’-9”W x 5’-0”H). Details similar to the 4-chord cantilever should be used in the design of these structures.

Note 2: The above sign areas are for one side only. Butterfly and Butterfly Truss structures can have double the total sign area listed with back-to-back signs mounted on each side of the structure.

39.1.8 Design Process

The design process for sign structures generally follows the process for bridge structures as detailed in chapter 6. There are some notable exceptions. First, the design of sign structures are usually initiated later in the overall process because they are dependent on a fairly established roadway plan. Second, a certain subset of sign structure types are permitted to be designed and detailed by a contractor, with other types requiring a department structural engineer (in-house or consultant) providing the design and detailing.

As outlined in 11-55-20.3 of the FDM, the Region initiates the sign structure design process by submitting to BOS an SSR. For *Contractor Designed* or *Standard Design* OSS types, as defined in 39.1.3, the Region or their consultant prepare final contract plans and submits via the structure e-submit process at least two months prior to PS&E. BOS must be notified if there are changes to the sign structure type after the SSR is submitted.

Region or consultant staff assemble final contract plans using the lead sheet templates and the *OSS Standard Design Drawings*, available on the BOS website under the Chapter 39 Bridge Standards - *LRFD Standardized Plans.* See 39.4.4 and 39.4.6 for more information on preparing standardized plans.

Involvement of a Department structural engineer in the design and detailing of individual sign structures is generally limited to *Non-standard* design types. If a Non-standard design is warranted, for reasons detailed in 39.4.5, then the design process follows the normal flow as defined in Chapter 6, requiring either BOS design staff or an engineering consultant provide a unique design and the final contract plans. Non-standard designs should make use of the OSS Standard Design Drawings where appropriate.

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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.

40.5.2 Selection Considerations

The selection of an overlay type is made considering several factors to achieve the desired extended service life. Several of these factors are provided in Table 40.5-1 and Table 40.5-2 to aid in the selection of an overlay for the preservation and rehabilitation of decks.

Overlay Type	Thin Polymer Overlay	Low Slump Concrete Overlay	Polyester Polymer Concrete Overlay (2)	Polymer Modified Asphaltic Overlay	Asphaltic Overlay (4)	Asphaltic Overlay with Membrane (2)
Overlay Life Span (years)	7 to 15	15 to 20	20 to 30	10 to 15	3 to 7	5 to 15
Traffic Impact (6)	< 1 day	7 days +/-	< 1 day	1-2 days	1-2 days	1-2 days
Overlay Costs ($/SF) (1)	$3 to $5	$4 to $7	$8 to $18	$10 to $22	$1 to $2	$5 to $8
Project Costs ($/SF) (1)	$4 to $8	$14 to $23	$10 to $30	$20 to $42	$4 to $10	$8 to $16
Overlay Minimum Thickness (Inches)	0.375	1.50	0.75	2.00	2.00	2.00
Wearing Surface Distress (delamination, spalls, or patches)	≤ 2%	≤ 25%	≤ 5%	≤ 25%	NA	≤ 25%
Deck Patch Material	Concrete (3), rapid set (2), or overlay mix	Overlay mix	Concrete (3), rapid set, or PPC	Concrete (3) or rapid set (2)	Concrete (3) or rapid set (2)	Concrete (3) or rapid set (2)
Typical Surface Preparation	Shot blast	Milled and shot blast (5)	Shot blast (5)	Sand blast	Water or air blast	Sand blast (5)
Overlay Finish	Aggregates	Tined	Tined and sanded	None	None	None

(1) Estimated costs based on CY2017 and is for informational pursues only. Overlay costs includes minimum overlay thickness and overlay placement costs. Project costs includes all structure associated costs (joint repairs, deck repairs, surface preparations, minimum overlay thickness). Costs do not include traffic control costs or other costs not captured on structure costs.
(2) Requires approval
(3) Portland cement concrete patch material may require a 28-day cure prior to overlay placement.
(4) Not eligible for federal funds
(5) 1 to 3/4-inch milling recommended for decks exposed longer than 10 years and not previously milled
(6) Estimated durations based on the overlay placement time to the minimum time until traffic can to be placed on the overlay. Durations do not include time for deck repairs or staging considerations.

Table 40.5-1
Overlay Selection Considerations

Overlay Type	**Advantages**	**Disadvantages**	**Notes**
Thin Polymer Overlay	• Minimal dead load • Minimal traffic disruptions • Seals the deck • Provides traction	• Requires a concrete age of at least 28 days • Requires decks with minimal defects and low chloride concentrations • Sensitive to moisture, temperature, and humidity at placement • Reflective cracking resistance concerns	
Low Slump Concrete Overlay	• Contractor familiarity and department experience • Long life span potential • Durable • Ease to accommodate grade differences and deficiencies	• Traffic disruptions • Additional dead load • High maintenance requirements • Railing height concerns • Susceptible to cracking • Specialized finishing equipment	• May require crack sealing the following year and periodically thereafter.
Polyester Polymer Concrete Overlay	• Minimal dead load • Minimal traffic disruptions • Seals the deck • Provides traction • Long life span potential • Durable • Low maintenance requirements	• High cost • Dedicated equipment • Limited usage in Wisconsin • Sensitive to moisture, temperature, and humidity at placement	• Requires BOS Prior-Approval
Polymer Modified Asphaltic Overlay	• Minimal traffic disruptions • Ease to construct • Can be used on more flexible structures (e.g. timber decks or timber slabs)	• High cost • Susceptible to permeability • Difficult to assess top of deck condition	• Contact region for availability • Minimal research has been performed on the durability of this system in Wisconsin
Asphaltic Overlay	• Low cost • Ease to construct • Ease to accommodate grade differences and deficiencies	• Short life span • Not eligible for federal funds • Overlay permeability • Difficult to assess top of deck condition	• Deck or bridge replacement should be programmed within 4 years
Asphaltic Overlay with Membrane	• Ease to construct • Minimal traffic disruptions • Long life span potential • Can be used on more flexible structures (e.g. PS box girders)	• Susceptible to permeability • Requires a membrane • Difficult to assess top of deck condition	• Currently under review • Requires BOS Prior-Approval

Table 40.5-2
Overlay Advantages, Disadvantages, and Notes

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. (Refer to **LRFD [5.13]** for current AASHTO guidance)

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