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5.1 Factors Governing Bridge Costs

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

1. Location - rural or urban, or remote regions

2. Type of crossing

3. Type of superstructure

4. Skew of bridge

5. Bridge on horizontal curve

6. Type of foundation

7. Type and height of piers

8. Depth and velocity of water

9. Type of abutment

10. Ease of falsework erection

11. Need for special equipment

12. Need for maintaining traffic during construction

13. Limit on construction time

14. Complex forming costs and design details

15. Span arrangements, beam spacing, etc.

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

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

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

5.3 Contract Unit Bid Prices

Refer to FDM 19-5-5 when preparing construction estimates and use the following estimating tools:

- Bid Express

- AASHTOWare Project Estimator

- [Estimating Tools](#) website

5.4 Bid Letting Cost Data

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

The square foot costs include all items shown on the structure plan except removing old structure. Costs also include a proportionate share of the project's mobilization, as well as structural approach slab costs, if applicable. However, square footage does not include the structural approach slabs, and is based on the length of the bridge from abutment to abutment. (It is realized that this yields a slightly higher square footage bridge cost for those bridges with structural approach slabs.)

5.4.1 2015 Year End Structure Costs

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

Table 5.4-1
Stream Crossing Structures

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

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

Hydraulic Data

100 YEAR FREQUENCY

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

2 YEAR FREQUENCY

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

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

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

(See Chapter 8 – Hydraulics for additional information)

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

6. Rehabilitation structure plans should use the same labeling convention as shown on the original structure plans when practical. Generally, this will include substructure labels (wings, abutments, piers, etc.) and girder numbers. This labeling convention is beneficial for inspection purposes.

6.2.2.4 Utilities

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

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

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

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

6.2.3 Distribution of Exhibits

This is a list of agencies that may need to be coordinated with. There may be other stakeholders that require coordination. Consult FDM Chapter 5 for more details on coordination requirements.

- Federal Highway Administration (FHWA)

   For unique structures, a copy of the finalized preliminary structure plans is forwarded by the BOS Design Supervisor to FHWA Division Bridge Engineer for review.

- Department of Natural Resources

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

- Railroad (FDM Chapter 17)

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

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

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

- Coast Guard (FDM)

- Regions

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

- Native American Tribal Governments

- Corps of Engineers

- Other governing municipalities

- State Historic Preservation Office

- Environmental Protection Agency

- Other DOTs

6.3 Final Plans

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

6.3.1 General Requirements

6.3.1.1 Drawing Size

Sheets are 11 inches deep from top to bottom and 17 inches long. A border line is provided on the sheet 5/8 inch from the left and right edges, and ¼ inch from top and bottom edges. Title blocks are provided on the first sheet for a signature and other required information. The following sheets contain the same information without provision for a signature.

6.3.1.2 Scale

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

6.3.1.3 Line Thickness

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

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

6.3.1.4 Lettering and Dimensions

All lettering is upper case. Lettering and dimensions are read from the bottom or righthand side and should be placed above the dimension lines. Notes and dimension text are approximately 0.06 inches high; view titles are approximately 0.10 inches high (based on a 11"x17" sheet). Dimensions are given in feet and inches. Elevations are given in decimal form to the nearest 0.01 of a foot. Always show two decimal places.

6.3.1.5 Notes

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

6.3.1.6 Standard Insert Drawings

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

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

6.3.1.7 Abbreviations

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

Abutment	ABUT.	Flange Plate	Fl. Pl.
Adjacent	ADJ.	Front Face	F.F.
Alternate	ALT.	Galvanized	GALV.
And	&	Gauge	GA.
Approximate	APPROX.	Girder	GIR.
At	@	Highway	HWY.
Back Face	B.F.	Horizontal	HORIZ.
Base Line	B/L	Inclusive	INCL.
Bench Mark	B.M.	Inlet	INL.
Bearing	BRG.	Invert	INV.
Bituminous	BIT.	Left	LT.
Cast-in-Place	C.I.P.	Left Hand Forward	L.H.F.
Centers	CTRS.	Length of Curve	L.
Center Line	C/L	Live Load	L.L.
Center to Center	C to C	Longitudinal	LONGIT.
Column	COL.	Maximum	MAX.
Concrete	CONC.	Minimum	MIN.
Construction	CONST.	Miscellaneous	MISC.
Continuous	CONT.	North	N.
Corrugated Metal Culvert Pipe	C.M.C.P.	Number	NO.
Cross Section	X-SEC.	Near Side, Far Side	N.S.F.S.
Dead Load	D.L.	Per Cent	%
Degree of Curve	D.	Plate	PL
Degree	°	Point of Curvature	P.C.
Diaphragm	DIAPH.	Point of Intersection	P.I.
Diameter	DIA.	Point of Tangency	P.T.
Discharge	DISCH.	Point on Curvature	P.O.C.
East	E.	Point on Tangent	P.O.T.
Elevation	EL.	Property Line	P.L.
Estimated	EST.	Quantity	QUAN.
Excavation	EXC.	Radius	R.
Expansion	EXP.	Railroad	R.R.
Fixed	F.	Railway	RY.
Reference	REF.	Station	STA.
Reinforcement	REINF.	Structural	STR.
Reinforced Concrete Culvert Pipe	R.C.C.P.	Substructure	SUBST.
Required	REQ'D.	Superstructure	SUPER.
Right	RT.	Surface	SURF.

Right Hand Forward	R.H.F.	Superelevation	S.E.
Right of Way	R/W	Symmetrical	SYM
Roadway	RDWY.	Tangent Line	TAN. LN.
Round	ø	Transit Line	T/L
Section	SEC.	Transverse	TRAN.
Shoulder	SHLD.	Variable	VAR.
Sidewalk	SDWK.	Vertical	VERT.
South	S.	Vertical Curve	V.C.
Space	SPA.	Volume	VOL.
Specification	SPEC	West	W.
Standard	STD.	Zinc Gauge	ZN. GA.

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

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

6.3.2 Plan Sheets

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

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

1. General Plan

2. Subsurface Exploration

3. Abutments

4. Piers

5. Superstructure and Superstructure Details

6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

See the BOS web page, CADD Resource Files, for the latest sheet borders to be used. Sheet borders are given for new bridges, rehabilitation projects and concrete box culverts. A superstructure replacement utilizing the existing substructure, bridge widenings, as well as

damaged girder replacements should use the sheet border for a new structure. See Chapter 40 - Bridge Rehabilitation for criteria as to when superstructure replacements are allowed.

1. Plan View

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

    a. Sufficient dimensions to layout structure in the field.

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

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

    On Structure Replacements

    Show existing structure in dashed-lines on Plan View.

2. Elevation View

    Same requirements as specified for preliminary plan except:

    a. Show elevation at bottom of all substructure units.

    b. Give estimated pile lengths where used.

3. Cross-Section View

    Same requirements as specified for preliminary plan except:

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

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

4. Grade Line

    Same requirements as specified for preliminary plan.

5. Design and Traffic Data

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

6. Hydraulic Information, if Applicable

7. Foundations

    Give soil/rock bearing capacity or pile capacity.

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

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

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

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

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

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

8. Estimated Quantities

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

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

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

9. General Notes

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

10. List of Drawings

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

11. Title Block

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

12. Professional Seal

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

6.3.2.1.1 Plan Notes for New Bridge Construction

1. Drawings shall not be scaled. Bar Steel Reinforcement shall be embedded 2" clear unless otherwise shown or noted.

2. All field connections shall be made with 3/4" diameter friction type high-tensile strength bolts unless shown or noted otherwise.

3. Slab falsework shall be supported on piles or the substructure unless an alternate method is approved by the Engineer.

4. The first or first two digits of the bar mark signifies the bar size.

5. The slope of the fill in front of the abutments shall be covered with heavy riprap and geotextile fabric Type 'HR' to the extent shown on sheet 1 and in the abutment details.

6. The slope of the fill in front of the abutments shall be covered with slope paving material to the extent shown on sheet 1 and in the abutment details.

7. The stream bed in front of the abutment shall be covered with riprap as shown on this sheet and in the abutment details.

8. The existing stream bed shall be used as the upper limits of excavation at the piers.

9. The existing ground line shall be used as the upper limits of excavation at the piers.

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

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

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

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

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

WisDOT policy item:

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

It is the responsibility of the design engineer to use original structure plans, as-built structure plans, shop drawings, field surveys and structure inspection reports as appropriate when producing rehabilitation structure plans of any type (bridges, retaining walls, box culverts, sign structures, etc.). **Note:** Older Milwaukee bridge plans used a baseline datum of 100.00. Add 580.60 to elevations using this datum. If uncertainty persists after reviewing available documentation, a field visit may be necessary by the design engineer.

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

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

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

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

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

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

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

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

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

6.3.2.2 Subsurface Exploration

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

1. Plan View

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

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

2. Elevation

    Show a centerline profile of existing ground elevation.

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

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

6.3.2.3 Abutments

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

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

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

1. Plan View

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

  b. Dimension wings in a direction parallel and perpendicular to the wing centerline. Wings should be numbered starting from the lower left corner and increasing in a clockwise order.

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

2. Elevation

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

  b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

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

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

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

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

1. Plan View

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

2. Elevation

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

3. Footing Plan

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

4. Bar Steel Listing and Details

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

6. Cross Section thru Column and Pier Cap

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

6.3.2.5 Superstructure

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

6.3.2.5.1 All Structures

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

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

    A separate deflection value for interior and exterior girders *may* be provided if the difference, accounting for load transfer between girders, warrants multiple values. A weighted distribution of composite dead load could be used for deflection purposes *only*. For example, an extremely large composite load over the exterior girder could be distributed as 40-30-30 percent to the exterior and first two interior girders respectively.

Use good engineering judgment to determine whether to provide separate deflection values for individual girder lines. In general, this is not necessary.

Indicate girder numbers about the centerline of bearing in each span. Girders should be numbered in increasing order from left to right in the cross-section view. For rehabilitation projects, indicate the existing girder numbers and assign new girder numbers in increasing order from left to right.

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

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

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

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

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

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

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

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

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

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

6.3.2.5.2 Steel Structures

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

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

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

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

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

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

6.3.2.5.3 Railing and Parapet Details

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

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

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

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

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

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

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

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

6.3.3.2 Box Culverts

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

1. Plan View

2. Longitudinal section

3. Section thru box

4. Wing elevations

5. Section thru wings

6. Section thru cutoff wall

7. Vertical construction joint

8. Bar steel clearance details

9. Header details

10. North point, 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. 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 reach consensus on which bid items to use when removing a particular structure. **The designer should not automatically defer to the recommendation from the initial DNR's Initial Review Letter (IRL), but should work with the WisDOT Regional Environmental Coordinator (REC), WisDOT Regional Stormwater & Erosion Control Engineer (SWECE) and DNR Transportation Liaison (TL), 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 Old Structure bid items should be selected for removals over waterways:

- Removing Structure Over Waterway Remove Debris (structure) 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 Minimal Debris (structure) 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 Debris Capture (structure) 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 Computation of Quantities

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

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

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

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

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

6.4.1 Excavation for Structures Bridges (Structure)

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

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

6.4.2 Granular Materials

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

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

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

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

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

Refer to 9.10 for additional information about granular materials.

6.4.3 Concrete Masonry Bridges

Show unit quantities (e.g. Pier 1) to the nearest 0.1 cubic yard. Show the total quantity to the nearest cubic yard. The unit quantities do not need to be adjusted so the sum of the unit quantities equals the total quantity. In computing quantities no deduction is made for metal reinforcement, floor drains, conduits and chamfers less than 2". Flanges of steel and prestressed girders projecting into the slab are deducted.

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

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

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

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

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

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

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

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

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

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

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

6.4.9 Piling Test Treated Timber (Structure)

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

6.4.10 Piling CIP Concrete (Size)(Shell Thickness), Piling Steel HP (Size)

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

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

- Steel Railing Type 'W' – CL end post to CL end post

- Tubular Railing Type 'H' – CL end plate to CL end plate

- Combination Railing Type '3T' – CL end post to CL end post + (2'-5") per railing

- Tubular Railing Type 'M' – CL end post to CL end post + (4'-6") per railing

- Combination Railing Type 'Type C1-C6' – CL end rail base plate to CL end rail base plate

- Tubular Steel Railing Type NY3&4 – CL end post to CL end post + (4'-10") per railing

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

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

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

Record this quantity to the nearest 1 cubic yard.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

For "Expansion Device" and "Expansion Device Modular", bid the items in lineal feet. The distance measured is from the outermost extent of the expansion device along the skew (do not include turn-ups into parapets or medians).

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

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

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

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

6.4.28 Removing Structure and Debris Containment

For work over roadways and railroads, "Removing Structure (structure)" is most typically used for complete or substantial removals. For work over waterways, one of the following Standard Specification bid items should be used for complete or substantial removals: Removing Structure Over Waterway Remove Debris (structure); Removing Structure Over Waterway Minimal Debris (structure); or Removing Structure Over Waterway Debris Capture (structure).

For work other than complete or substantial removals, a Removing Structure (structure) bid item may not be required.

Use Debris Containment (structure) bid items, **only as needed** based on the significance, extent, or location of the removal.

See 6.3.3.8 for additional information on Removing Structure and Debris Containment bid items.

Bid as each.

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.30 Steel Diaphragms (Structure)

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

6.4.31 Welded Stud Shear Connectors X -Inch

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

6.4.32 Concrete Masonry Seal

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

6.4.33 Geotextile Fabric Type

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

6.4.34 Concrete Adhesive Anchors

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

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

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

6.4.36 Piling Steel Sheet Temporary

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

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

6.4.37 Temporary Shoring

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

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

6.4.38 Concrete Masonry Deck Repair

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

6.4.39 Sawing Pavement Deck Preparation Areas

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

6.4.40 Removing Bearings

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

6.4.41 Ice Hot Weather Concreting

Used to provide a mechanism for payment of ice during hot weather concreting operations. See FDM 19-5-3.2 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.

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

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

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

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

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

The QA/QC plan shall also include the following items:

- Identification of a lead QA/QC Structures Program contact
- Identification of the QA/QC plan and procedures implementation date
- A statement indicating that the independent design check will be performed by an individual other than the designer, and the independent plan check will be performed by an individual other than the drafter.

Provisions for periodic reviews and update of the QA/QC plan with a frequency no less than 5 years; or as needed due to changes in the firm's personnel or firm's processes or procedures; or as requested by BOS.A QA/QC verification summary sheet is required as part of every final structure plan submittal, demonstrating that the QA/QC plan and procedures were followed for that structure. The QA/QC verification summary sheet shall include the signoff or initialing by each individual that performed the tasks (design, checking, plan review, technical review, etc.) documented in the QA/QC plan and procedures. The summary sheet must be submitted with the final structure plans as part of the e-submit process.

Consultants' QA/QC plans and verification summary sheets may be subject to periodic reviews by BOS. These reviews are intended to assess compliance with BOS requirements listed above.

The list of consultant firms eligible to provide structural design services to WisDOT may be accessed using the link below:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/plan-submittal.aspx

6.5.1 Approvals, Distribution, and Work Flow

Consultant	Meet with Regional Office and/or local units of government to determine need.
	Prepare Structure Survey Report including recommendation of structure type.
Geotechnical Consultant	Make site investigation and prepare Subsurface Investigation Report.
Consultant	Submit hydrology report via Esubmit or as an email attachment to the supervisor of the Consultant Review and Hydraulics Unit. Submit 60 days prior to preliminary plan submittal.
	Prepare preliminary plans according to 6.2.
	Coordinate with Region and other agencies per 6.2.3.
	Submit preliminary plans, SSR and supporting documents via e-submit for review and approval of type, size and location.
Structures Design Section	Record project information in HSIS.
	Review hydraulics for Stream Crossings.
	Review Preliminary Plan. A minimum of 30 days to review preliminary plans should be expected.
	Coordinate with other agencies per 6.2.3.
	Return preliminary plans and comments from Structures Design Section and other appropriate agencies to Consultant with a copy to the Regional Office.
	Forward Preliminary Plan and Hydraulic Data to DNR.
Consultant	Modify preliminary plan as required, and provide explanation for preliminary comments not incorporated in final plan.
	Prepare and complete final design and plans for the specified structure.
	Write special provisions.
	At least **two months** in advance of the PS&E date, submit the required final design documents via e-submit per 6.5.3.

Structures Design Section	Determine which final plans will be reviewed and perform quality assurance review as applicable.
	For final plans that are reviewed, return comments to Consultant and send copy to Regional Office, including FHWA as appropriate.
Consultant	Modify final plans and specifications as required.
	Submit modified final plans via e-submit as required.
Structures Design Section	Review modified final plans as applicable.
	Sign final plans and send performance evaluation form to Region and Consultant.
Geotechnical Consultant	At time of PSE, transmit gINT boring logs, soils laboratory testing summary and data sheets, and Soil Reports to the emails provided in the Soils and Subsurface Investigations section of Two/Three Party Design Contract Special Provisions.
Bureau of Project Development	Prepare final accepted structure plans for pre-development contract administration.
Consultant	If a plan change is needed after being advertised but before being let, an addendum is required per FDM 19-22-1 and 19-22 Attachment 1.2.
Structures Design Section	Review structure addendum as applicable.
	Sign structure addendum.
Bureau of Project Development	Distributes structure addendum to bidders.
Consultant	If a plan change is required after being let, a post-let revision is required per 6.5.5.
Structures Design Section	Review post-let revision as applicable.
	Stamp post-let revision plan as accepted.
	Delivers revised plan to DOT construction team for distribution.

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Preliminary Plan Requirements

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

1. Hydrology Report

2. Structure Survey Report

3. Preliminary plan, including log borings shown on the subsurface exploration sheet

4. Evaluation of subsurface investigation report

5. Contour map

6. Plan and profile, and typical section for roadway approaches

7. Hydraulic/Sizing Report (see Chapter 8 - Hydraulics) and hydraulic files are required for stream crossing structures

8. County map showing location of new and/or existing structures and FEMA map

9. Any other information or drawings which may influence location, layout or design of structure, including DNR initial review letter and photographs

6.5.3 Final Plan Requirements

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

1. Final Drawings

2. Design and Quantity Computations

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

3. Special Provisions covering unique items not in the Standard Specifications or Standardized Special Provisions (STSP).

4. QA/QC Verification Sheet

5. Inventory Data Sheet

6. Bridge Load Rating Summary Form

7. LRFD Input File (Excel ratings spreadsheet)

8. On-Time Improvement Form

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

    - If the first version of a final structure plan is submitted after the deadline of two months prior to the PSE date.

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

6.5.4 Addenda

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

6.5.5 Post-Let Revisions

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

6.5.6 Local-Let Projects

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

6.6 Structures Data Management and Resources

6.6.1 Structures Data Management

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Table 6.6-1
Various Inspection Reports
**   HSI – Highway Structures Information System – The electronic file where bridge data is stored for future use.

6.6.2 Resources

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

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/default.aspx

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

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

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

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

http://bridges.transportation.org

https://www.arema.org/

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9.1 General

The *Wisconsin Standard Specifications for Highway and Structure Construction* (hereafter referred to as *Standard Specifications*) contains references to *ASTM Specifications* or *AASHTO Material Specifications* which provide required properties and testing standards for materials used in highway structures. The service life of a structure is dependent upon the quality of the materials used in its construction as well as the method of construction. This chapter highlights applications of materials for highway structures and their properties.

In cases where proprietary products are experimentally specified, special provisions are written which provide material properties and installation procedures. Manufacturer's recommendations for materials, preparation and their assistance during installation may also be specified.

Materials that are proposed for incorporation into highway structure projects performed under the jurisdiction of the Wisconsin Department of Transportation (WisDOT) may be approved or accepted by a variety of procedures:

- Laboratory testing of materials that are submitted or samples randomly selected.

- Inspection and/or testing at the source of fabrication or manufacture.

- Inspection and/or testing in the field by WisDOT regional personnel.

- Manufacturer's certificate of compliance and/or manufacturer's certified report of test or analysis, either as sole documentation for acceptance or as supplemental documentation.

- Inspection, evaluation and testing in the normal course of project administration of material specifications.

- Some products are on approved lists or from fabricators, manufacturers, and certified sources approved by WisDOT. Lists of approved suppliers, products, and certified sources are located at:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

The *Wisconsin Construction and Materials Manual* (CMM) contains a description of procedures for material testing and acceptance requirements in Chapter 8, Section 45. Materials, unless otherwise permitted by the specifications, cannot be incorporated in the project until tested and approved by the proper authority.

9.7 Miscellaneous Materials

Several types of materials are being used as part of a bridge deck protective system. Epoxy coated reinforcing steel, mentioned earlier, is part of this system. Some of these materials or products, are experimental and are placed on specific structures and then monitored and evaluated. A list of materials or products that are part of a bridge deck protective system and are currently used or under evaluation are:

- Galvanized or stainless steel reinforcing bars

- Waterproofing membrane with bituminous concrete overlay

- Microsilica modified concrete or polymer impregnated concrete

- Low slump concrete overlays

- Low-viscosity crack sealer

- Cathodic protection systems with surface overlays

Other materials or products used on highway structures are:

- Downspouts for Type GC and H drains may be fabricated from fiberglass conforming to ASTM D2996, Grade 1, Class A.

- Elastomeric bearing pads (non-laminated) are primarily used with prestressed "I" girders at fixed abutments and piers and at semi-expansion abutments. They are also used with prestressed "slab and box" sections at all supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.

- Elastomeric bearing pads (laminated) are primarily used with prestressed "I" girders at expansion supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.

- Preformed fillers are placed vertically in the joint between wing and diaphragm in A1 and A5 abutments, in the joint between wing and barrel in box culverts and at expansion joints in concrete cast-in-place retaining walls. Preformed fillers are placed along the front top surface of A1 and A5 abutments, along the outside top surfaces of fixed piers and under flanges between elastomeric bearing pad (non-laminated) and top edge of support. Cork filler is placed vertically on semi-expansion abutments. The requirements for fillers are described in Section 502.2.7 of the *Standard Specifications*.

- Polyethelene sheets are placed on the top surface of semi-expansion abutments to allow movement of the superstructure across the bearing surface. They are also placed between the structural approach slab and the subgrade, and the approach slab and its footing.

- Rubberized waterproofing membranes are used to seal horizontal and vertical joints at the backface of abutments, culverts and concrete cast-in-place retaining walls. See Section 516.2.3 of the *Standard Specifications*.

- Non-staining gray non-bituminous joint sealer is used to seal exposed surfaces of preformed fillers placed in joints as described above. It is also used to place a seal around exposed surfaces of plates used at deflection joints and around railing base plates. The requirement for this joint sealer is referenced in Section 502.2.9 of the *Standard Specifications*.

- Plastic plates may be used at deflection joints in sidewalks and parapets.

- Preformed Fabric, Class A, has been used as a bearing pad under steel bearings. The requirement for this material is given in Section 506.2.8.4 of the *Standard Specifications*.

- Neoprene strip seals are used in single cell and multi-cell (modular) expansion devices.

- Teflon sheets are bonded to steel plates in Type A-T expansion bearings. The requirements for these sheets are found in Section 506.2.8.3 of the *Standard Specifications*.

- Asphalt panels are used on railroad structures to protect the rubber membrane on top of the steel ballast plate from being damaged by the ballast. The requirements for these panels are in the "*Special Provisions*".

- Geotextile fabric is used for drainage filtration, and under riprap and box culverts. This fabric consists of sheets of woven or non-woven synthetic polymers or nylon. Type DF is used for drainage filtration in the pipe underdrain detail placed behind abutments and walls. The fabric allows moisture to drain to the pipe while keeping the backfill from migrating into the coarse material and then into the pipe. Type DF is also used behind abutments or walls that retain soil with backing planks between or behind piling and also for some of the walls detailed in Chapter 14 – Retaining Walls. This fabric will allow moisture to pass through the fabric and the joints in the walls without migration of the soil behind the wall. Type R or HR is placed below riprap and will keep the soil beneath it from being washed away. Type C is placed under breaker run when it is used under box culverts. The requirements for these fabrics are found in Section 645.2 of the *Standard Specifications*.

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Figure 12.4-2
Wings Parallel to Roadway and Wing Wall Angle < 90°

For wing wall angle, α < 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.
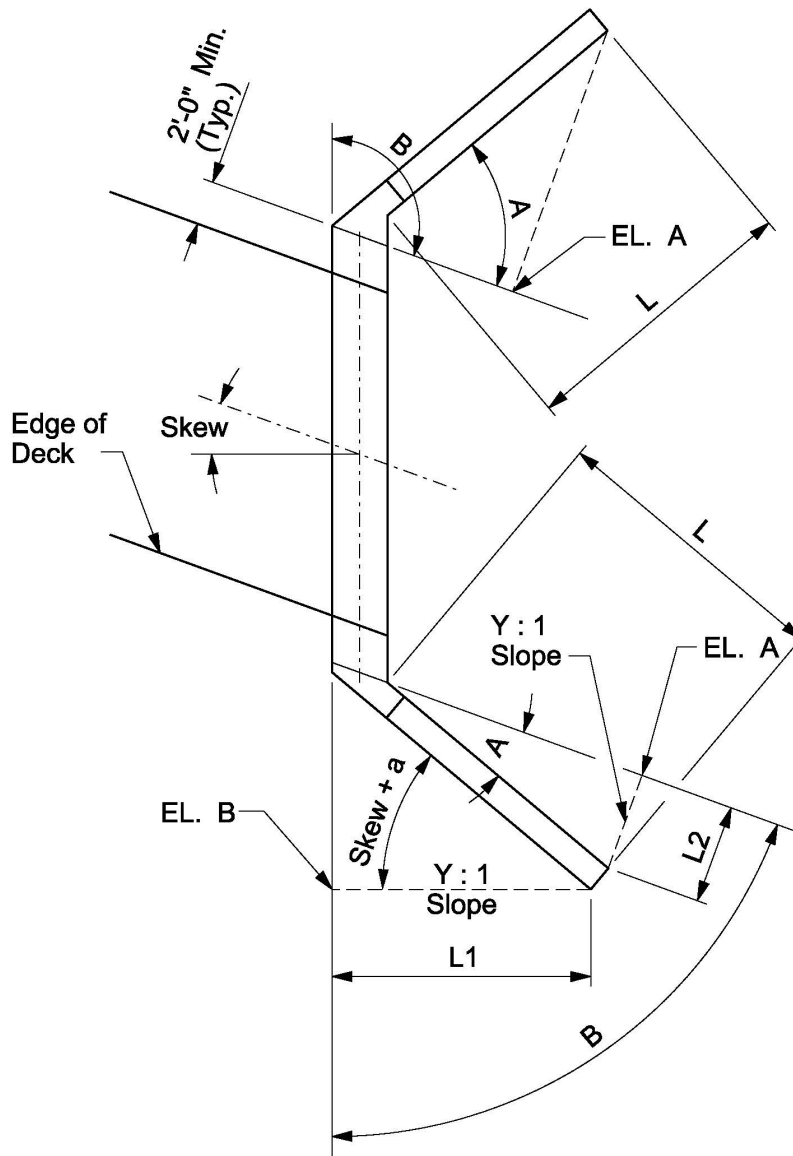
Figure 12.4-3
Wings Not Parallel to Roadway and Equal Slopes

For angle B ≥ 90°:

$$L1 + L2 = (EL. A - EL. B)(Y)$$

$$\cos(a - Skew) = \frac{L1}{L}$$

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WisDOT policy item:

Designs for bridge piers adjacent to roadways with a design speed ≤ 40 mph need not consider the provisions of **LRFD [3.6.5]**.

If the design speed of a roadway adjacent to a pier is > 40 mph and the pier is not protected by a TL-5 barrier, embankment or adequate offset, the pier columns/shaft, *only*, shall be strengthened to comply with **LRFD [3.6.5]**. For a multi-column pier the minimum size column shall be 3x4 ft rectangular or 4 ft diameter (consider clearance issues and/or the wide cap required when using 4 ft diameter columns). Solid shaft and hammerhead pier shafts are considered adequately sized.

All multi-columned piers require a minimum of three columns. If a pier cap consists of two or more segments each segment may be supported by two columns. If a pier is constructed in stages, two columns may be used for the temporary condition.

The vertical reinforcement for the columns/shaft shall be the greater of what is required by design (not including the Extreme Event II loading) or a minimum of 1.0% of the gross concrete section (total cross section without deduction for rustications less than or equal to 1-1/2" deep) to address the collision force for the 3x4 ft rectangular and 4 ft diameter columns.

For the 3x4 ft rectangular columns, use double #5 stirrups spaced at 6" vertically as a minimum. For the 4 ft diameter columns, use #4 spiral reinforcement (smooth bars) spaced vertically at 6" as a minimum. Hammerhead pier shafts shall have, as a minimum, the horizontal reinforcement as shown on the Standards.

See Standard for Multi-Columned Pier with Rectangular Columns for an acceptable design to meet **LRFD [3.6.5]**.

WisDOT exception to AASHTO:

The vessel collision load, CV, in **LRFD [3.14]** will not be applied to every navigable waterway of depths greater than 2'. For piers located in navigable waterways, the engineer shall contact the WisDOT project manager to determine if a vessel collision load is applicable.

13.5 Load Application

When determining pier design forces, a thorough understanding of the load paths for each load is critical to arriving at loads that are reasonable per *AASHTO LRFD*. The assumptions associated with different pier, bearing and superstructure configurations are also important to understand. This section provides general guidelines for the application of forces to typical highway bridge piers.

13.5.1 Loading Combinations

Piers are designed for the Strength I, Strength III, Strength V and Extreme Event II load combinations as specified in **LRFD [3.4.1]**. Reinforced concrete pier components are also checked for the Service I load combination. Load factors for these load combinations are presented in Table 13.5-1. See 13.10 for loads applicable to pile bents and pile encased piers.

Load Combination	Load Factor										
	DC		DW		LL+IM	WA	WS	WL	FR	TU	IC
	Max.	Min.	Max.	Min.	BR					CR	CT
Limit State					CE					SH	CV
Strength I	1.25	0.90	1.50	0.65	1.75	1.00	0.00	0.00	1.00	0.5*	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.00	0.00	1.00	0.5*	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	1.00	1.00	1.00	0.5*	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
Extreme Event II	1.00	1.00	1.00	1.00	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Table 13.5-1
Load Factors

 * Values based on using gross moment of inertia for analysis **LRFD [3.4.1]**

13.5.2 Expansion Piers

See 13.4 for additional guidance regarding the application of specific loads.

Transverse forces applied to expansion piers from the superstructure include loads from one-half of the adjacent span lengths, and are applied at the location of the transverse load.

For expansion bearings other than elastomeric, longitudinal forces are transmitted to expansion piers through friction in the bearings. These forces, other than temperature, are based on loading one-half of the adjacent span lengths, with the maximum being no greater than the maximum friction force (dead load times the maximum friction coefficient of a sliding bearing). See 27.2.2 to determine the bearing friction coefficient.   The longitudinal forces are applied at the bearing elevation.

Skin reinforcement on the side of the cap, shall be determined as per **LRFD [5.6.7]**. This reinforcement shall not be included in any strength calculations.

See 13.1 and 13.2.1 for further requirements specific to this pier type.

13.7 Hammerhead Pier Cap Design

WisDOT policy item:

Hammerhead pier caps shall be designed using the strut-and-tie method **LRFD [5.8.2]**.

The strut-and-tie method (STM) is simply the creation of an internal truss system used to transfer the loads from the bearings through the pier cap to the column(s). This is accomplished through a series of concrete "struts" that resist compressive forces and steel "ties" that resist tensile forces. These struts and ties meet at nodes **LRFD [5.8.2.1]**. See Figure 13.7-1 for a basic strut-and-tie model that depicts two bearing reactions transferred to two columns. STM is used to determine internal force effects at the strength and extreme event limit states.
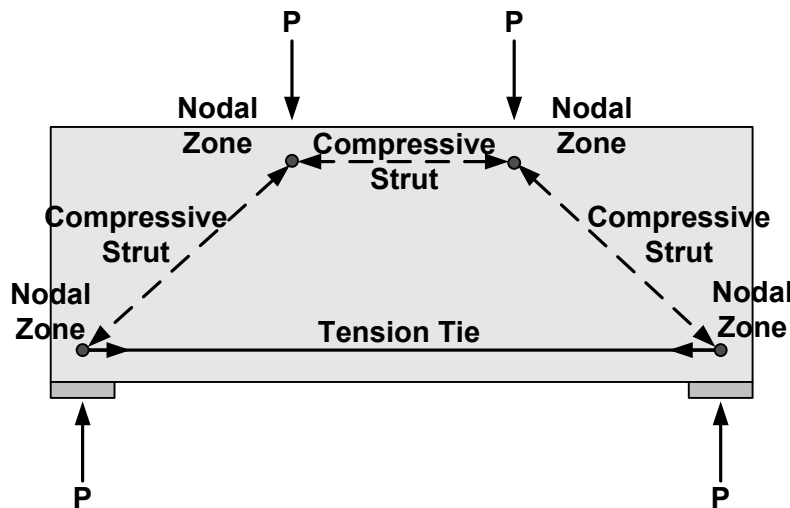
Figure 13.7-1
Basic Strut-and-Tie Elements

Strut-and-tie models are based on the following assumptions:

- The tension ties yield before the compressive struts crush.

- External forces are applied at nodes.

- Forces in the struts and ties are uniaxial.

- Equilibrium is maintained.

- Prestressing of the pier is treated as a load.

The generation of the model requires informed engineering judgment and is an iterative, graphical procedure. The following steps are recommended for a strut-and-tie pier cap design.

      a. Over one-half of the piles in the footings must be in compression for the Strength limit states. The section properties used in analysis are based only on the piles in compression. Pile and footing (pile cap) design is based on the Strength limit state and also the check for overall stability per **LRFD [10.7.3.1]**. Service limit state check of crack control is not required per 13.11. The 600 kip collision load need not be checked per 13.4.10.

      b. Piles may be designed for upward forces provided an anchorage device, sufficient to transfer the load, is provided at the top of the pile. Provide reinforcing steel to resist the tension stresses at the top of the footing.

3. Same as spread footing.

4. Same as spread footing.

5. The minimum number of piles per footing is four.

6. Pile footings on seals are analyzed above the seal. The only effect of the seal is to reduce the pile resistance above the seal by the portion of the seal weight carried by each pile.

7. If no seal is required but a cofferdam is required, design the piles to use the minimum required batter. This reduces the cofferdam size necessary to clear the battered piles since all piles extend above water to the pile driver during driving.

8. The pile footing reinforcing steel is determined from the flexural requirements of **LRFD [5.6.3]**. The design moment and shear are determined from the force of the piles which act outside of the section being considered. The weight of the footing and the soil above the footing are used to reduce the magnitude of the bending moment and shear force.

9. Shear resistance is determined by the following two methods:

      a. Two-way action

      The summation of the pile forces outside the critical perimeter lines placed at a distance d/2 from the face of the column (where d equals the effective footing depth) determines the shear force. The shear resistance is influenced by the concrete strength, the footing depth and the critical perimeter length. The critical perimeter length is 2 (L + d + W + d) for rectangular columns and π(2R + d) for round columns. The critical perimeter location for pile footings with rectangular columns is illustrated in Figure 13.11-3.
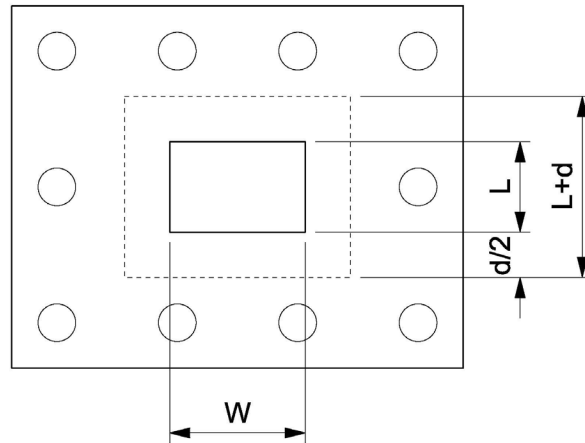
Figure 13.11-3
Critical Perimeter Location for Pile Footings

If the center of a pile falls on a line, then one-half of the pile force is assumed to act on each side of the line.

b. One-way action

The summation of the pile forces located within the area enclosed by the footing edges and a line at distance "d" from the face of the column determines the shear force, as illustrated in Figure 13.11-2. The shear resistance is influenced by the concrete strength, the footing depth and the entire footing width or length. If the center of a pile falls on a line, then one-half of the pile force is assumed to act on each side of the line.

10. The weight of the footing and soil above the areas is used to reduce the shear force.

11. The bottom layer of reinforcing steel is placed directly on top of the piles.

13.11.4 Continuous Footings

Continuous footings are used in pier frames of two or more columns when the use of isolated footings would result in a distance of less than 4'-6" between edges of adjacent footings. They are designed for the moments and shears produced by the frame action of the pier and the soil pressure under the footing.

The soil pressure or pile load under the footing is assumed to be uniform. The soil pressures or pile loads are generally computed only from the vertical column loads along with the soil and footing dead load. The moments at the base of the column are ignored for soil or pile loads.

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E13-1 Hammerhead Pier Design Example

This example shows design calculations conforming to the ***AASHTO LRFD Bridge Design Specifications (Ninth Edition - 2020)*** as supplemented by the *WisDOT Bridge Manual* The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice. Updates to strut and tie procedures will be coming soon, to this example. Please follow the current AASHTO Spec. when designing these elements. The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-1.1 Obtain Design Criteria

This pier is designed for the superstructure as detailed in example **E24-1**. This is a two-span steel girder stream crossing structure. Expansion bearings are located at the abutments, and fixed bearings are used at the pier.
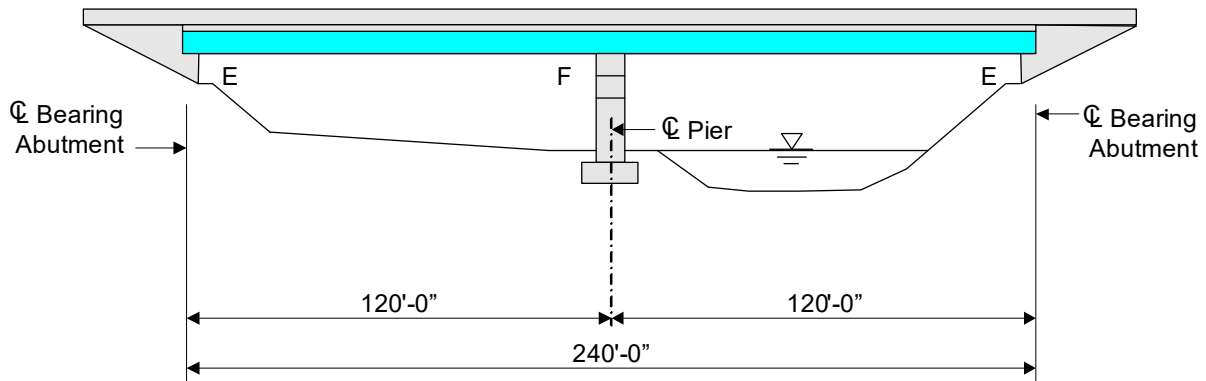
Figure E13-1.1-1
Bridge Elevation
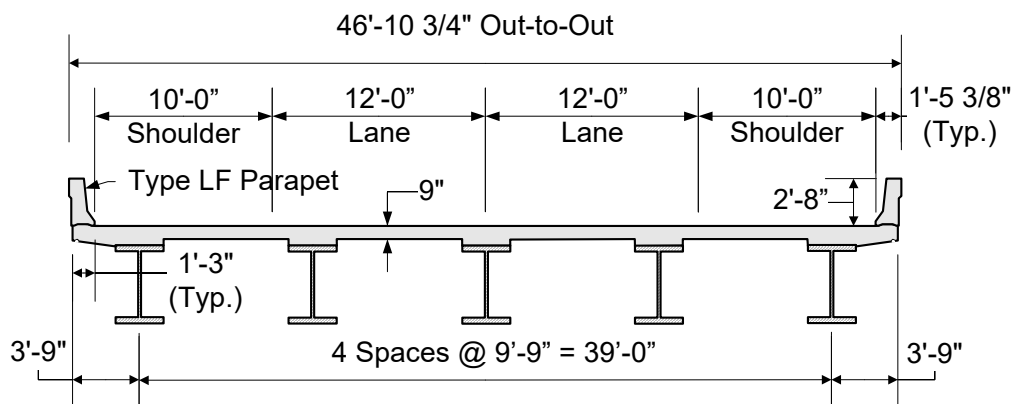
Figure E13-1.1-2
Bridge Cross Section

E13-1.1.1 Material Properties:

$w_c := 0.150$	unit weight of concrete, kcf
$f'_c := 3.5$	concrete 28-day compressive strength, ksi
$f_y := 60$	reinforcement yield strength, ksi

E13-1.1.2 Reinforcing Steel Cover Requirements:

All cover dimensions listed below are in accordance with **LRFD [Table 5.10.1-1]** and are shown in <u>inches</u>.

$Cover_{cp} := 2.5$	Pier cap
$Cover_{co} := 2.5$	Pier column
$Cover_{ft} := 2.0$	Footing top cover
$Cover_{fb} := 6.0$	Footing bottom cover, based on standard pile projection

E13-1.2 Relevant Superstructure Data

$w_{deck} := 46.50$	Deck Width, ft
$w_{roadway} := 44.0$	Roadway Width, ft
$ng := 5$	Number of Girders
$S := 9.75$	Girder Spacing, ft
$DOH := 3.75$	Deck Overhang, ft (Note that this overhang exceeds the limits stated in Chapter 17.6.2. WisDOT practice is to limit the overhang to 3'-7".)
$N_{spans} := 2$	Number of Spans
$L := 120.0$	Span Length, ft
$skew := 0$	Skew Angle, degrees
$H_{super} = 8.46$	Superstructure Depth, ft
$H_{brng} := 6.375$	Bearing Height, in (Fixed, Type A)
$W_{brng} := 18$	Bearing Width, in
$L_{brng} := 26$	Bearing Length, in
$\mu_{max} := 0.10$	Max. Coefficient of Friction of Abutment Expansion Bearings
$\mu_{min} := 0.06$	Min. Coefficient of Friction of Abutment Expansion Bearings

E13-1.2.1 Girder Dead Load Reactions

Unfactored Dead Load Reactions, <u>kips</u>

$$
DLR_{int} := \begin{pmatrix}
\text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\
\text{"Beam"} & 7.00 & 34.02 \\
\text{"Misc"} & 1.23 & 4.73 \\
\text{"Deck"} & 46.89 & 178.91 \\
\text{"Parapet"} & 6.57 & 24.06 \\
\text{"FWS"} & 7.46 & 27.32
\end{pmatrix}
\qquad
DLR_{ext} := \begin{pmatrix}
\text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\
\text{"Beam"} & 7.00 & 34.02 \\
\text{"Misc"} & 0.83 & 3.15 \\
\text{"Deck"} & 48.57 & 185.42 \\
\text{"Parapet"} & 6.57 & 24.06 \\
\text{"FWS"} & 7.46 & 27.32
\end{pmatrix}
$$

Abutment Reactions:

$$AbutRint_{DC} = 61.69 \quad \text{kips} \qquad\qquad AbutRext_{DC} = 62.97 \quad \text{kips}$$

$$AbutRint_{DW} = 7.46 \quad \text{kips} \qquad\qquad AbutRext_{DW} = 7.46 \quad \text{kips}$$

Pier Reactions:

$$Rint_{DC} = 241.72 \quad \text{kips} \qquad\qquad Rext_{DC} = 246.65 \quad \text{kips}$$

$$Rint_{DW} = 27.32 \quad \text{kips} \qquad\qquad Rext_{DW} = 27.32 \quad \text{kips}$$

E13-1.2.2 Live Load Reactions per Design Lane

Unfactored Live Load Reactions, <u>kips</u>

$$
LLR := \begin{pmatrix}
\text{"LoadType"} & \text{"Abut"} & \text{"Pier"} \\
\text{"Vehicle"} & 64.72 & 114.17 \\
\text{"Lane"} & 32.76 & 89.41
\end{pmatrix}
$$

These loads are per design lane and do not include dynamic load allowance. The pier reactions are controlled by the 90% (Truck Pair + Lane) loading condition. The reactions shown include the 90% factor.

E13-1.3 Select Preliminary Pier Dimensions

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. For this design example, a single column (hammerhead) pier was chosen.

Since the LRFD Specifications do not have standards regarding maximum or minimum dimensions for a pier cap, column, or footing, the designer should base the preliminary pier dimensions on WisDOT specific standards, previous designs, and past experience. The pier cap, however, must be wide enough to accommodate the bearings.

Figures E13-1.3-1 and E13-1.3-2 show the preliminary dimensions selected for this pier design example.
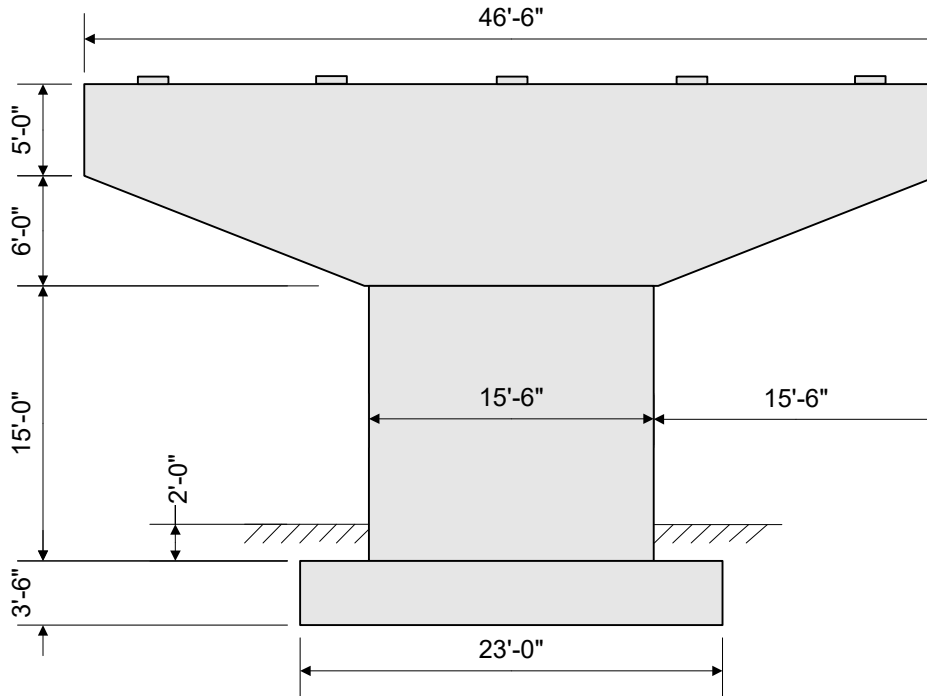
Figure E13-1.3-1
Preliminary Pier Dimensions - Front Elevation
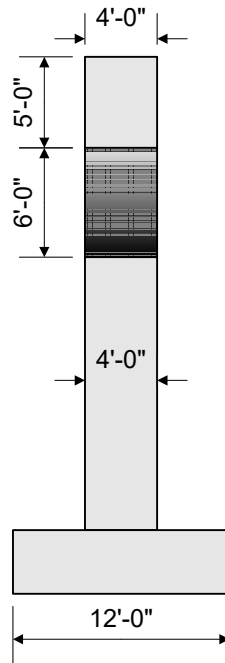
Figure E13-1.3-2
Preliminary Pier Dimensions - End Elevation

Pier Geometry Definitions (feet):

$L_{cap} := 46.5$ $L_{col} := 15.5$ $L_{ftg} := 23$

$W_{cap} := 4$ $W_{col} := 4$ $W_{ftg} := 12$

$H_{cap} := 11$ $H_{col} := 15$ $H_{ftg} := 3.5$

$H_{cap\_end} := 5$ $D_{soil} := 2$ Soil depth above footing, feet

$L_{oh} := 15.5$ $\gamma_{soil} := 0.120$ Unit weight of soil, kcf

E13-1.4 Compute Dead Load Effects

Once the preliminary pier dimensions are selected, the corresponding dead loads can be computed in accordance with **LRFD [3.5.1]**. The pier dead loads must then be combined with the superstructure dead loads.

Exterior girder dead load reactions (DC and DW): $Rext_{DC} = 246.65$ kips

                                                        $Rext_{DW} = 27.32$ kips

Interior girder dead load reactions (DC and DW): $\boxed{\text{Rint}_{DC} = 241.72}$ kips

$\boxed{\text{Rint}_{DW} = 27.32}$ kips

Pier cap dead load:

$$DL_{Cap} := w_c \cdot W_{cap} \cdot \left[2 \cdot \left(\frac{H_{cap\_end} + H_{cap}}{2} \right) \cdot L_{oh} + H_{cap} \cdot L_{col} \right]$$

$$= 0.150 \cdot 4 \cdot \left(2 \cdot \frac{5+11}{2} \cdot 15.5 + 11 \cdot 15.5 \right) \qquad \boxed{DL_{Cap} = 251.1} \qquad \text{kips}$$

Pier column dead load:

$$DL_{col} := w_c \cdot W_{col} \cdot H_{col} \cdot L_{col}$$

$$= 0.150 \cdot 4 \cdot 15 \cdot 15.5 \qquad \boxed{DL_{col} = 139.5} \qquad \text{kips}$$

Pier footing dead load:

$$DL_{ftg} := w_c \cdot W_{ftg} \cdot H_{ftg} \cdot L_{ftg}$$

$$= 0.150 \cdot 12 \cdot 3.5 \cdot 23 \qquad \boxed{DL_{ftg} = 144.9} \qquad \text{kips}$$

In addition to the above dead loads, the weight of the soil on top of the footing must be computed. The two-foot height of soil above the footing was previously defined. Assuming a unit weight of soil at 0.120 kcf in accordance with **LRFD [Table 3.5.1-1]** :

$$EV_{ftg} := \gamma_{soil} \cdot D_{soil} \cdot \left(W_{ftg} \cdot L_{ftg} - W_{col} \cdot L_{col} \right)$$

$$= 0.120 \cdot 2 \cdot (12 \cdot 23 - 4 \cdot 15.5) \qquad \boxed{EV_{ftg} = 51.36} \qquad \text{kips}$$

E13-1.5 Compute Live Load Effects

For the pier in this design example, the maximum live load effects in the pier cap, column and footing are based on either one, two or three lanes loaded (whichever results in the worst force effect). Figure E13-1.5-1 illustrates the lane positions when three lanes are loaded.

The positioning shown in Figure E13-1.5-1 is determined in accordance with **LRFD [3.6.1].** The first step is to calculate the number of design lanes, which is the integer part of the ratio of the clear roadway width divided by 12 feet per lane. Then the lane loading, which occupies ten feet of the lane, and the HL-93 truck loading, which has a six-foot wheel spacing and a two-foot clearance to the edge of the lane, are positioned within each lane to maximize the force effects in each of the respective pier components.
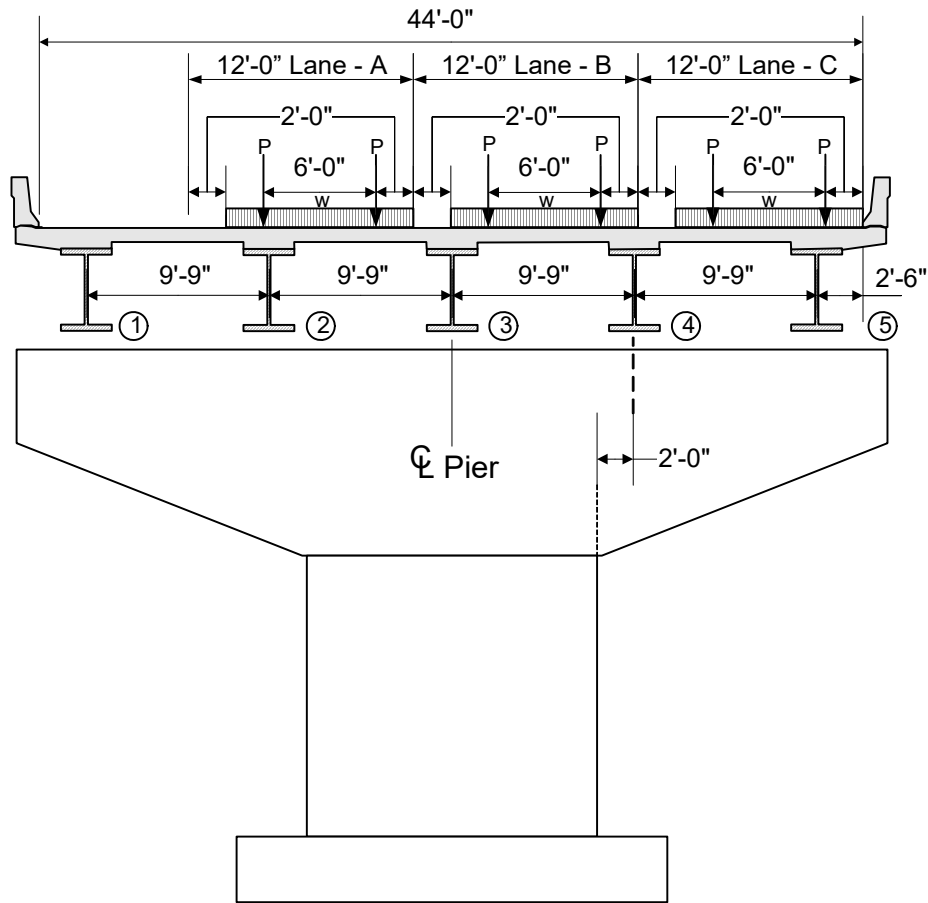
Figure E13-1.5-1
Pier Live Loading

N   = maximum number of design lanes that the bridge can accommodate
$w_{roadway}$ = roadway width between curbs, ignoring any median strip
W   = design lane width

W := 12   feet

$w_{roadway}$ = 44   feet

$N := \dfrac{w_{roadway}}{W}$ N = 3.67

                                         N = 3   design lanes

The unfactored girder reactions for lane load and truck load are obtained from the superstructure analysis and are as shown in E13-1.1.3.2. These reactions do not include dynamic load allowance and are given on a per lane basis (i.e., distribution factor = 1.0). Also, the reactions include the ten percent reduction permitted by the Specifications for interior pier reactions that result from longitudinally loading the superstructure with a truck pair in conjunction with lane loading **LRFD [3.6.1.3.1]**.

Live load reactions at Pier (w/o distribution):

$R_{truck} = 114.17$ kips

$R_{lane} = 89.41$ kips

$IM := 0.33$ Dynamic load allowance, IM from **LRFD [Table 3.6.2.1-1]**

The values of the unfactored concentrated loads which represent the girder truck pair load reaction per wheel line in Figure E13-1.5-1 are:

$$P_{wheel} := \frac{R_{truck}}{2} \cdot (1 + IM)$$ $P_{wheel} = 75.92$ kips

The value of the unfactored uniformly distributed load which represents the girder lane load reaction in Figure E13-1.5-1 is computed next. This load is transversely distributed over ten feet and is not subject to dynamic load allowance, **LRFD [3.6.2.1]**.

$$W_{lane} := \frac{R_{lane}}{10}$$ $W_{lane} = 8.94$ $\dfrac{kips}{ft}$

The next step is to compute the reactions due to the above loads at each of the five bearing locations. This is generally carried out by assuming the deck is pinned (i.e., discontinuous) at the interior girder locations but continuous over the exterior girders. Solving for the reactions is then elementary. The computations for the reactions with only Lane C loaded are illustrated below as an example. The subscripts indicate the bearing location and the lane loaded to obtain the respective reaction:

$$R_{5\_c} := \frac{P_{wheel} \cdot (4.25 + 10.25) + W_{lane} \cdot 10 \times 7.25}{9.75}$$ $R_{5\_c} = 179.4$ kips

$$R_{4\_c} := P_{wheel} \cdot 2 + W_{lane} \cdot 10 - R_{5\_c}$$ $R_{4\_c} = 61.86$ kips

The reactions at bearings 1, 2 and 3 with only Lane C loaded are zero. Calculations similar to those above yield the following live load reactions with the remaining lanes loaded. All reactions shown are in <u>kips</u>.

Lane A Loaded	Lane B Loaded	Lane C Loaded
$R_{5\_a} := 0.0$	$R_{5\_b} := 0.0$	$R_{5\_c} = 179.4$
$R_{4\_a} := 0.0$	$R_{4\_b} = 123.66$	$R_{4\_c} = 61.86$
$R_{3\_a} = 72.31$	$R_{3\_b} = 117.56$	$R_{3\_c} := 0.0$
$R_{2\_a} = 164.67$	$R_{2\_b} := 0.0$	$R_{2\_c} := 0.0$
$R_{1\_a} = 4.27$	$R_{1\_b} := 0.0$	$R_{1\_c} := 0.0$

E13-1.6 Compute Other Load Effects

Other load effects that will be considered for this pier design include braking force, wind loads, and temperature loads.

For simplicity, buoyancy, stream pressure, ice loads and earthquake loads are not included in this design example.

E13-1.6.1 Braking Force

Since expansion bearings exist at the abutments, the entire longitudinal braking force is resisted by the pier.

In accordance with **LRFD [3.6.4]**, the braking force per lane is the greater of:

  25 percent of the axle weights of the design truck or tandem

  5 percent of the axle weights of the design truck plus lane load

  5 percent of the axle weights of the design tandem plus lane load

The total braking force is computed based on the number of design lanes in the same direction. It is assumed in this example that this bridge is likely to become one-directional in the future. Therefore, any and all design lanes may be used to compute the governing braking force. Also, braking forces are not increased for dynamic load allowance in accordance with **LRFD [3.6.2.1]**. The calculation of the braking force for a single traffic lane follows:

25 percent of the design truck:

$$BRK_{trk} := 0.25 \cdot (32 + 32 + 8)$$ $\boxed{BRK_{trk} = 18}$ kips

25 percent of the design tandem:

$$BRK_{tan} := 0.25 \cdot (25 + 25)$$ $\boxed{BRK_{tan} = 12.5}$ kips

5 percent of the axle weights of the design truck plus lane load:

$$BRK_{trk\_lan} := 0.05 \cdot [(32 + 32 + 8) + (0.64 \times 2 \cdot L)]$$ $\boxed{BRK_{trk\_lan} = 11.28}$ kips

5 percent of the axle weights of the design tandem plus lane load:

$$BRK_{tan\_lan} := 0.05 \cdot [(25 + 25) + (0.64 \times 2 \cdot L)]$$ $\boxed{BRK_{tan\_lan} = 10.18}$ kips

Use:

$$BRK := max(BRK_{trk}, BRK_{tan}, BRK_{trk\_lan}, BRK_{tan\_lan})$$ $\boxed{BRK = 18}$ kips per lane

LRFD [3.6.4] states that the braking force is applied along the longitudinal axis of the bridge at a distance of six feet above the roadway surface. However, since the skew angle is zero for this design example and the bearings are assumed incapable of transmitting longitudinal moment, the braking force will be applied at the top of bearing elevation. For bridges with skews, the component of the braking force in the transverse direction would be applied six feet above the roadway surface.

This force may be applied in either horizontal direction (back or ahead station) to cause the maximum force effects. Additionally, the total braking force is typically assumed equally distributed among the bearings:

$$BRK_{brg} := \frac{BRK}{5}$$
$\boxed{BRK_{brg} = 3.6}$ kips per bearing per lane

The moment arm about the base of the column is:

$$H_{BRK} := H_{col} + H_{cap} + \frac{H_{brng}}{12}$$
$\boxed{H_{BRK} = 26.53}$ feet

E13-1.6.2 Wind Load from Superstructure

Prior to calculating the wind load on the superstructure, the structure must be checked for aeroelastic instability, **LRFD [3.8.3]**. If the span length to width or depth ratio is greater than 30, the structure is considered wind-sensitive and design wind loads should be based on wind tunnel studies. This wind load applies to Strength III, Strength V, and Service I.

$\boxed{H_{par} = 2.67}$ Parapet height, feet

Span Length (L): $\boxed{L = 120}$ feet

$Width := w_{deck}$ $\boxed{Width = 46.5}$ feet

$Depth := H_{super} - H_{par}$ $\boxed{Depth = 5.79}$ feet

$\boxed{\frac{L}{Width} = 2.58}$ < 30 OK $\boxed{\frac{L}{Depth} = 20.72}$ < 30 OK

Since the span length to width and depth ratios are both less than 30, the structure does not need to be investigated for aeroelastic instability.

To compute the wind load on the superstructure, the area of the superstructure exposed to the wind must be defined. For this example, the exposed area is the total superstructure depth, (H_{super}), multiplied by length tributary to the pier. Due to expansion bearings at the abutment, the transverse length tributary to the pier is not the same as the longitudinal length.

The superstructure depth includes the total depth from the top of the barrier to the bottom of the girder. Included in this depth is any haunch and/or depth due to the deck cross-slope.

Once the total depth is known, (H_{super}), the exposed wind area can be calculated and the design wind pressure applied.

The total depth was previously computed in Section E13-1.1 and is as follows: $\boxed{H_{super} = 8.46}$ feet

For this two-span bridge example, the tributary length for wind load on the fixed pier in the transverse direction is one-half of each adjacent span:

$$L_{windT} := \frac{L + L}{2} \qquad\qquad \boxed{L_{windT} = 120} \qquad \text{feet}$$

In the longitudinal direction, the tributary length is the entire bridge length due to the expansion bearings at the abutments:

$$L_{windL} := L \cdot 2 \qquad\qquad \boxed{L_{windL} = 240} \qquad \text{feet}$$

The transverse wind area is:

$$A_{wsuperT} := H_{super} \cdot L_{windT} \qquad\qquad \boxed{A_{wsuperT} = 1015} \qquad \text{ft}^2$$

The longitudinal wind area is: $\boxed{A_{wsuperL} = \blacksquare}$

$$A_{wsuperL} := H_{super} \cdot L_{windL} \qquad\qquad\qquad\qquad \text{ft}^2$$

The design wind pressures applied to the superstructure are shown in Section 13.4.4. To calculate the wind pressure to be used for Strength III, the value of (Z) must be calculated to select the value of (K_Z) in **LRFD [Table C3.8.1.2.1-1]**.

The value of (Z) at the pier is:

$$Z_{pier} := H_{col} - D_{soil} + H_{cap} + H_{super} + \frac{H_{brng}}{12} \qquad\qquad \boxed{Z_{pier} = 32.99} \qquad \text{feet}$$

Therefore, the average value of (Z) will be less than 33 feet, and because the Wind Exposure Category C applies to this structure, use:

$$K_Z := 1.0 \qquad \text{; therefore } Psup_{III} = 0.044 \text{ ksf}$$

Because the maximum height above low ground or water level to top of structure is (Z_{pier}), which is 33 feet, and individual span lengths are less than 150 feet, the values for transverse and longitudinal wind forces may be calculated using the simplified method in Section 13.4.4.1.

Strength III:

$$Psup_{transIII} := 0.044 \quad \text{ksf} \qquad \text{(transverse)}$$

$$Psup_{longitIII} := 0.011 \quad \text{ksf} \qquad \text{(longitudinal)}$$

Strength V:

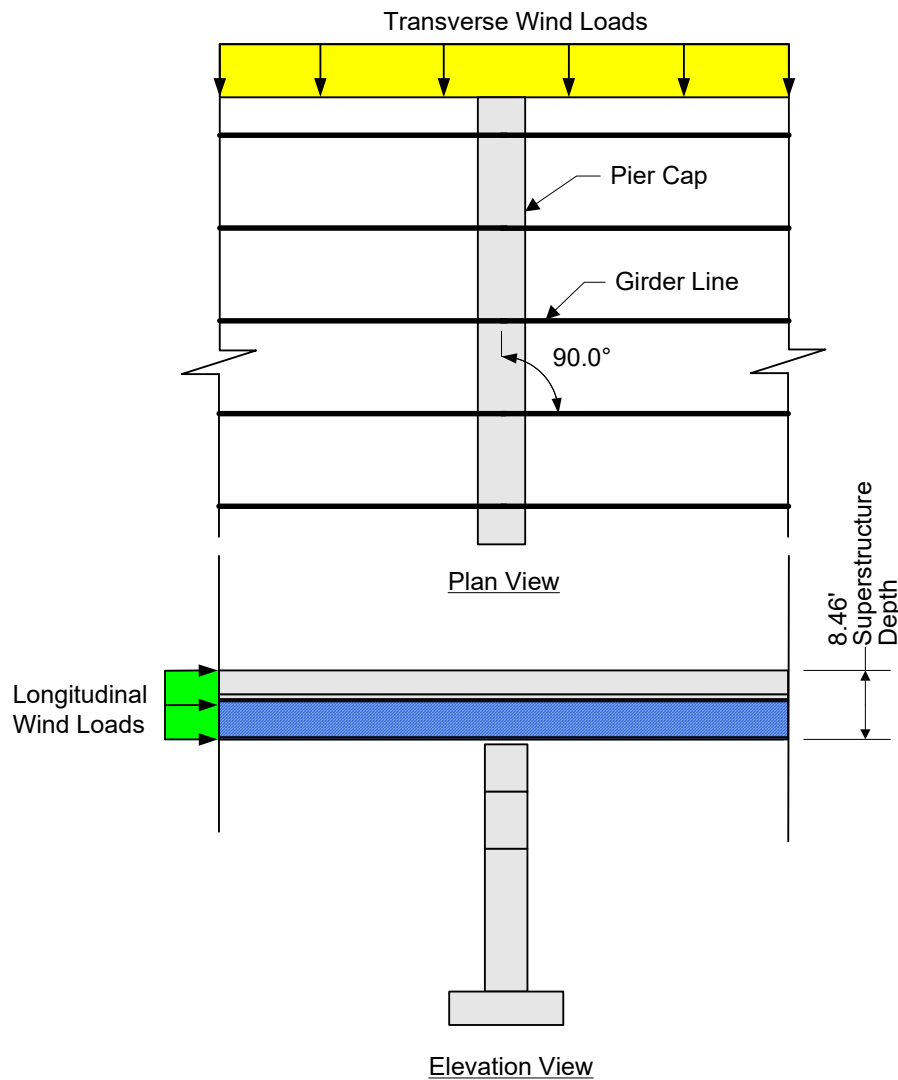$Psup_{transV} := 0.021$ ksf (transverse)

$Psup_{longitV} := 0.006$ ksf (longitudinal)

Service I:

$Psup_{transI} := 0.016$ ksf (transverse)

$Psup_{longitI} := 0.004$ ksf (longitudinal)

Transverse Wind Loads

Pier Cap

Girder Line

90.0°

Plan View

8.46' Superstructure Depth

Longitudinal Wind Loads

Elevation View

Figure E13-1.6-1
Application of Wind Load

The transvers and longitudinal superstructure wind loads acting on the pier (girders) are:

<u>Strength III:</u>

$WS_{suptrnsIII} := A_{wsuperT} \cdot Psup_{transIII}$

$\boxed{WS_{suptrnsIII} = 44.68}$ kips

$WS_{suplngIII} := A_{wsuperL} \cdot Psup_{longitIII}$

$\boxed{WS_{suplngIII} = 22.34}$ kips

<u>Strength V:</u>

$WS_{suptrnsV} := A_{wsuperT} \cdot Psup_{transV}$

$\boxed{WS_{suptrnsV} = 21.32}$ kips

$WS_{suplngV} := A_{wsuperL} \cdot Psup_{longitV}$

$\boxed{WS_{suplngV} = 12.18}$ kips

<u>Service I:</u>

$WS_{suptrnsI} := A_{wsuperT} \cdot Psup_{transI}$

$\boxed{WS_{suptrnsI} = 16.25}$ kips

$WS_{suplngI} := A_{wsuperL} \cdot Psup_{longitI}$

$\boxed{WS_{suplngI} = 8.12}$ kips

The total longitudinal wind loads (WS_{suplng}) shown above is assumed to be divided equally among the bearings. In addition, the load at each bearing is assumed to be applied at the top of the bearing. These assumptions are consistent with those used in determining the bearing forces due to the longitudinal braking force.

The horizontal force (WS_L) at each bearing due to the longitudinal wind loads on the superstructure are:

$$WS_{L\_III} := \frac{WS_{suplngIII}}{5}$$

$\boxed{WS_{L\_III} = 4.47}$ kips

$$WS_{L\_V} := \frac{WS_{suplngV}}{5}$$

$\boxed{WS_{L\_V} = 2.44}$ kips

$$WS_{L\_I} := \frac{WS_{suplngI}}{5}$$

$\boxed{WS_{L\_I} = 1.62}$ kips

The transverse wind loads ($WS_{suptrns}$) shown above are also assumed to be equally divided among the bearings but are applied at the mid-depth of the superstructure.

The horizontal force (WS_T) at each bearing due to the transverse wind loads on the superstructure are:

$$WS_{T\_III} := \frac{WS_{suptrnsIII}}{5}$$

$$WS_{T\_III} = 8.94 \quad \text{kips}$$

$$WS_{T\_V} := \frac{WS_{suptrnsV}}{5}$$

$$WS_{T\_V} = 4.26 \quad \text{kips}$$

$$WS_{T\_I} := \frac{WS_{suptrnsI}}{5}$$

$$WS_{T\_I} = 3.25 \quad \text{kips}$$

These horizontal forces (WS_T) are shown in Figure E13-1.6-2

For calculating the resulting moment effect on the column, the moment arm about the base of the column for transverse and longitudinal wind forces are:

$$H_{WSlong} := H_{col} + H_{cap} + \frac{H_{brng}}{12}$$

$$H_{WSlong} = 26.53 \quad \text{feet}$$

$$H_{WStrns} := H_{col} + H_{cap} + \frac{H_{brng}}{12} + \frac{H_{super}}{2}$$

$$H_{WStrns} = 30.76 \quad \text{feet}$$

However, the transverse load also applies a moment to the pier cap. This moment, which acts about the centerline of the pier cap, induces vertical loads at the bearings as illustrated in Figure E13-1.6-2. The computations for these vertical forces are presented below.
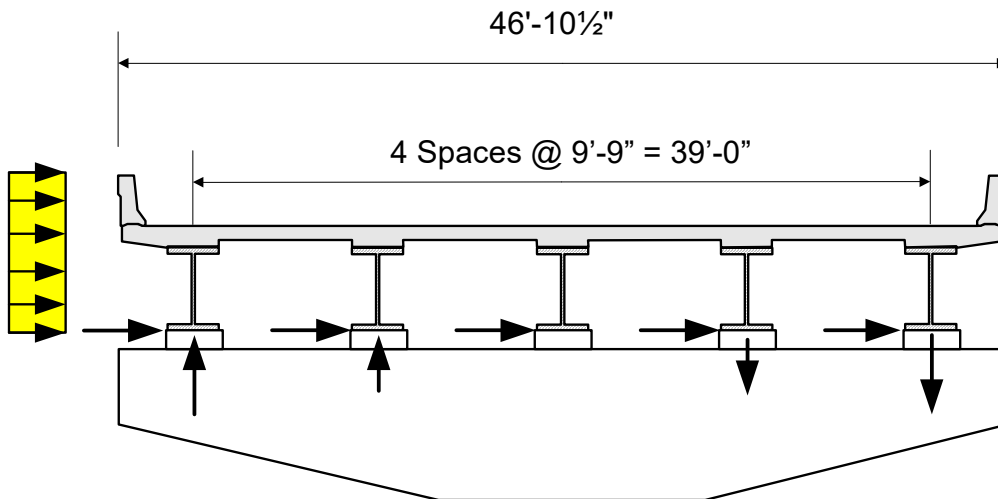
Figure E13-1.6-2
Transverse Wind Loads at Pier Bearings from Wind on Superstructure

Transverse Moments on the Pier Cap:

$$M_{trnsIII} := WS_{suptrnsIII} \cdot \left(\frac{H_{super}}{2}\right)$$

$M_{trnsIII} = 189.02$ kip-ft

$$M_{trnsV} := WS_{suptrnsV} \cdot \left(\frac{H_{super}}{2}\right)$$

$M_{trnsV} = 90.22$ kip-ft

$$M_{trnsI} := WS_{suptrnsI} \cdot \left(\frac{H_{super}}{2}\right)$$

$M_{trnsI} = 68.74$ kip-ft

Moment of Inertia for the Girder Group:

$$I = \Sigma A \cdot y^2$$

$A = 1$ $I_1 = I_5$ $I_2 = I_4$ $I_3 = 0$

$S = 9.75$ feet (girder spacing)

$$I_{girders} := 2 \cdot (S + S)^2 + 2 \cdot S^2$$

$$= 2 \cdot (9.75 + 9.75)^2 + 2 \cdot 9.75^2$$

$I_{girders} = 950.63$ ft²

$$Reaction = \frac{Moment \cdot y}{I}$$

Vertical Forces at the Bearings:

$$RWS1\_5_{trnsIII} := \frac{M_{trnsIII} \cdot (S + S)}{I_{girders}}$$

$RWS1\_5_{trnsIII} = 3.88$ kips

$$RWS1\_5_{trnsV} := \frac{M_{trnsV} \cdot (S + S)}{I_{girders}}$$

$RWS1\_5_{trnsV} = 1.85$ kips

$$RWS1\_5_{trnsI} := \frac{M_{trnsI} \cdot (S + S)}{I_{girders}}$$

$RWS1\_5_{trnsI} = 1.41$ kips

The loads at bearings 1 and 5 are equal but opposite in direction. Similarly for bearings 2 and 4:

$$RWS2\_4_{trnsIII} := \frac{M_{trnsIII} \cdot S}{I_{girders}}$$

$RWS2\_4_{trnsIII} = 1.94$ kips

$$RWS2\_4_{trnsV} := \frac{M_{trnsV} \cdot S}{I_{girders}}$$

$$\boxed{RWS2\_4_{trnsV} = 0.93} \quad \text{kips}$$

$$RWS2\_4_{trnsl} := \frac{M_{trnsl} \cdot S}{I_{girders}}$$

$$\boxed{RWS2\_4_{trnsl} = 0.70} \quad \text{kips}$$

Finally, by inspection: $\boxed{RWS3_{trns} = 0}$ kips

These vertical forces (RWS) are shown in Figure E13-1.6-2

E13-1.6.2.1 Vertical Wind Load

The vertical (upward) wind load is calculated by multiplying a <u>0.020 ksf</u> vertical wind pressure by the out-to-out bridge deck width as described in Section 13.4.4.4. It is applied at the windward quarter-point of the deck only for limit states that do not include wind on live load (Strength III). The wind load is then multiplied by the tributary length, which is one-half of each adjacent span.

From previous definitions:

$\boxed{w_{deck} = 46.5}$ ft $\boxed{L_{windT} = 120}$ ft

The total vertical wind load is then:

$$WS_{vert} := 0.02 \left(w_{deck} \right) \cdot \left(L_{windT} \right)$$

$$\boxed{WS_{vert} = 111.6} \quad \text{kips}$$
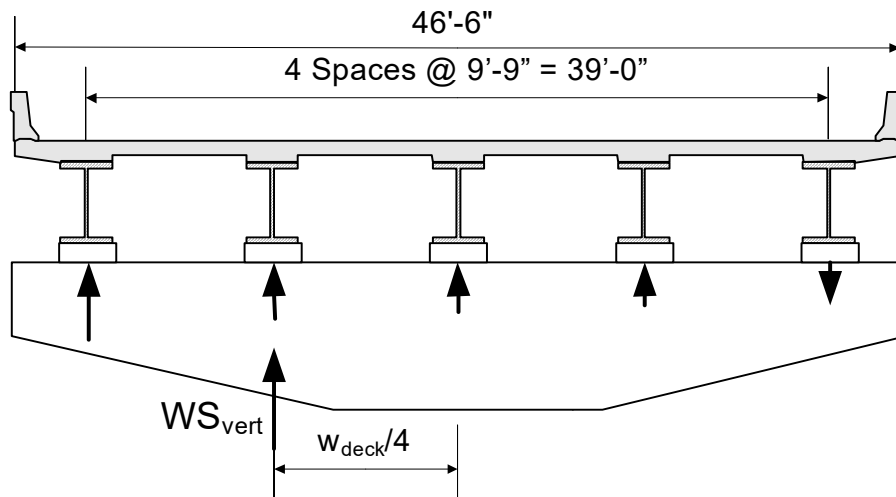
Figure E13-1.6-3
Vertical Wind Loads at Pier Bearings from Wind on Superstructure

This load causes a moment about the pier centerline. The value of this moment is:

$$M_{WS\_vert} := WS_{vert} \cdot \frac{w_{deck}}{4} \qquad\qquad \boxed{M_{WS\_vert} = 1297.35} \quad \text{kip-ft}$$

The vertical loads at the bearings are computed as: $\boxed{S = 9.75}$ feet (girder spacing)

$$RWS_{vert1} := \frac{-WS_{vert}}{5} + \frac{M_{WS\_vert} \cdot (2 \cdot S)}{I_{girders}} \qquad \boxed{RWS_{vert1} = 4.29} \quad \text{kips}$$

$$RWS_{vert2} := \frac{-WS_{vert}}{5} + \frac{M_{WS\_vert} \cdot S}{I_{girders}} \qquad \boxed{RWS_{vert2} = -9.01} \quad \text{kips}$$

$$RWS_{vert3} := \frac{-WS_{vert}}{5} \qquad\qquad\qquad \boxed{RWS_{vert3} = -22.32} \quad \text{kips}$$

$$RWS_{vert4} := \frac{-WS_{vert}}{5} - \frac{M_{WS\_vert} \cdot S}{I_{girders}} \qquad \boxed{RWS_{vert4} = -35.63} \quad \text{kips}$$

$$RWS_{vert5} := \frac{-WS_{vert}}{5} - \frac{M_{WS\_vert} \cdot 2 \cdot S}{I_{girders}} \qquad \boxed{RWS_{vert5} = -48.93} \quad \text{kips}$$

Where a negative value indicates a vertical upward load. These loads only apply to <u>Strength III.</u>

E13-1.6.2.2 Wind Load on Vehicles

The representation of wind pressure acting on vehicular traffic is given by **LRFD [3.8.1.3]** as a uniformly distributed line load. This load is applied both transversely and longitudinally. For the transverse and longitudinal loadings, the total force in each respective direction is calculated by multiplying the appropriate component by the length of structure tributary to the pier. Similar to the superstructure wind loading, the longitudinal length tributary to the pier differs from the transverse length. As shown in E13-1.6.2, the criteria for using the simplified method in Section 13.4.4.3 has been met, and the transverse and longitudinal loads are calculated as shown below and are to be applied simultaneously. This wind load applies to Strength V and Service I.

$\boxed{L_{windT} = 120}$ feet $\boxed{L_{windL} = 240}$ feet

$P_{LLtrans} := 0.100$ klf

$P_{LLlongit} := 0.040$ klf

$WL_{trans} := L_{windT} \cdot P_{LLtrans} \qquad\qquad \boxed{WL_{trans} = 12} \quad \text{kips}$

$WL_{long} := L_{windL} \cdot P_{LLlongit} \qquad\qquad \boxed{WL_{long} = 9.6} \quad \text{kips}$

The wind on vehicular live loads shown above are applied to the bearings in the same manner as the wind load from the superstructure. That is, the total transverse and longitudinal load is equally distributed to each bearing and applied at the top of the bearing.

The horizontal forces (WL_T, WL_L) at each bearing due to wind load on vehicles are:

$$WL_{T\_V} := \frac{WL_{trans}}{5}$$ 　　　　　　$\boxed{WL_{T\_V} = 2.4}$ 　　kips

$$WL_{T\_I} := \frac{WL_{trans}}{5}$$ 　　　　　　$\boxed{WL_{T\_I} = 2.4}$ 　　kips

$$WL_{L\_V} := \frac{WL_{long}}{5}$$ 　　　　　　$\boxed{WL_{L\_V} = 1.92}$ 　　kips

$$WL_{L\_I} := \frac{WL_{long}}{5}$$ 　　　　　　$\boxed{WL_{L\_I} = 1.92}$ 　　kips

In addition, the transverse load acting six feet above the roadway applies a moment to the pier cap. This moment induces vertical reactions at the bearings. The values of these vertical reactions are given below. The computations for these reactions are not shown but are carried out as shown in E13-1.6.2. The only difference is that the moment arm used for calculating the moment is equal to (H_{super} - H_{par} + 6.0 feet).

$$Mom_{arm} := H_{super} - H_{par} + 6$$ 　　　　$\boxed{Mom_{arm} = 11.79}$ 　　feet

Vertical Forces at the Bearings:

$\boxed{RWL1\_5_{trns} = 2.9}$ 　　kips

$\boxed{RWL2\_4_{trns} = 1.45}$ 　　kips

$\boxed{RWL3_{trns} = 0}$ 　　kips

For calculating the resulting moment effect on the column, the moment arm about the base of the column is:

$$H_{WLlong} := H_{col} + H_{cap} + \frac{H_{brng}}{12}$$ 　　　$\boxed{H_{WLlong} = 26.53}$ 　　feet

$$H_{WLtrns} := H_{col} + H_{cap} + \frac{H_{brng}}{12} + (H_{super} - H_{par} + 6)$$ 　$\boxed{H_{WLtrns} = 38.32}$ 　　feet

E13-1.6.3 Wind Load on Substructure

The design wind pressure applied directly to the substructure units are shown in Section 13.4.4. As stated in E13-1.6.2, for Strength III the value of K_Z = 1.0. For simplicity, apply the same pressure in the transverse and longitudinal directions for Strength III, V and Service I.

Strength III:

$P_{subIII} := 0.054$ ksf   (transverse/longitudinal)

Strength V:

$P_{subV} := 0.026$ ksf   (transverse/longitudinal)

Service I:

$P_{subI} := 0.020$ ksf   (transverse/longitudinal)

In accordance with Section 13.4.4.2, the transverse and longitudinal wind forces calculated from these wind pressures acting on the corresponding exposed areas are to be applied simultaneously. These loads shall also act simultaneously with the superstructure wind loads.

Plan View
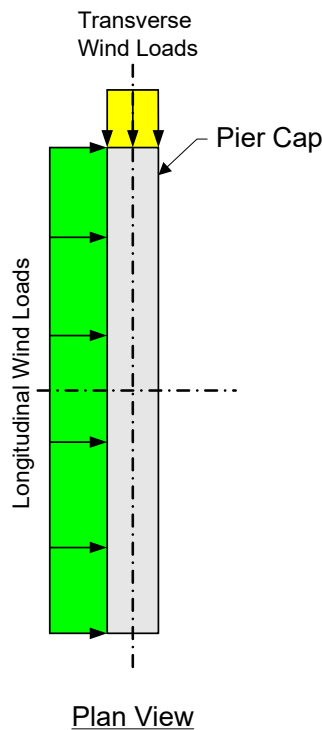
Figure E13-1.6-4
Wind Pressure on Pier

What follows is an example of the calculation of the wind loads acting directly on the pier. For simplicity, the tapers of the pier cap overhangs will be considered solid. The column height exposed to wind is the distance from the ground line (which is two feet above the footing) to

the bottom of the pier cap.

Component areas of the pier cap:

$$A_{capLong} := (L_{cap}) \cdot (H_{cap})$$
$\boxed{A_{capLong} = 511.5}$ ft^2

$$A_{capTrans} := (W_{cap}) \cdot (H_{cap})$$
$\boxed{A_{capTrans} = 44}$ ft^2

Component areas of the pier column:

$$A_{colLong} := (L_{col}) \cdot (H_{col} - D_{soil})$$
$\boxed{A_{colLong} = 201.5}$ ft^2

$$A_{colTrans} := (W_{col}) \cdot (H_{col} - D_{soil})$$
$\boxed{A_{colTrans} = 52}$ ft^2

The transverse and longitudinal substructure wind loads acting on the pier are:

Strength III:

$$WS_{subLIII} := P_{subIII} \cdot (A_{capLong} + A_{colLong})$$
$\boxed{WS_{subLIII} = 38.5}$ kips

$$WS_{subTIII} := P_{subIII} \cdot (A_{capTrans} + A_{colTrans})$$
$\boxed{WS_{subTIII} = 5.18}$ kips

Strength V:

$$WS_{subLV} := P_{subV} \cdot (A_{capLong} + A_{colLong})$$
$\boxed{WS_{subLV} = 18.54}$ kips

$$WS_{subTV} := P_{subV} \cdot (A_{capTrans} + A_{colTrans})$$
$\boxed{WS_{subTV} = 2.50}$ kips

Service I:

$$WS_{subLI} := P_{subI} \cdot (A_{capLong} + A_{colLong})$$
$\boxed{WS_{subLI} = 14.26}$ kips

$$WS_{subTI} := P_{subI} \cdot (A_{capTrans} + A_{colTrans})$$
$\boxed{WS_{subTI} = 1.92}$ kips

The point of application of these loads will be the centroid of the loaded area of each face, respectively.

$$H_{WSsubL} := \frac{A_{capLong} \cdot \left(H_{col} + \dfrac{H_{cap}}{2} \right) + A_{colLong} \cdot \left(\dfrac{H_{col} - 2}{2} + 2 \right)}{A_{capLong} + A_{colLong}}$$

$\boxed{H_{WSsubL} = 17.11}$ feet

$$H_{WSsubT} := \frac{A_{capTrans} \cdot \left(H_{col} + \dfrac{H_{cap}}{2}\right) + A_{colTrans} \cdot \left(\dfrac{H_{col} - 2}{2} + 2\right)}{A_{capTrans} + A_{colTrans}}$$

$H_{WSsubT} = 14$ feet

E13-1.6.4 Temperature Loading (Superimposed Deformations)

In this particular structure, with a single pier centered between two abutments that have identical bearing types, the temperature force is based on assuming a minimum coefficient of expansion at one abutment and the maximum at the other using only dead load reactions. This force acts in the longitudinal direction of the bridge (either back or ahead station) and is equally divided among the bearings. Also, the forces at each bearing from this load will be applied at the top of the bearing.

The abutment girder Dead Load reactions from E13-1.2.1 are as follows:

$AbutRint_{DC} = 61.69$ $AbutRext_{DC} = 62.97$

$AbutRint_{DW} = 7.46$ $AbutRext_{DW} = 7.46$

$\mu_{min} = 0.06$

$\mu_{max} = 0.1$

$\Delta\mu := \mu_{max} - \mu_{min}$ $\Delta\mu = 0.04$

$F_{TU} := \Delta\mu \cdot \left[3 \cdot \left(AbutRint_{DC} + AbutRint_{DW}\right) + 2 \cdot \left(AbutRext_{DC} + AbutRext_{DW}\right)\right]$

$F_{TU} = 13.93$ kips

The resulting temperature force acting on each bearing is:

$$TU_{BRG} := \frac{F_{TU}}{5}$$

$TU_{BRG} = 2.79$ kips

The moment arm about the base of the column is:

$$H_{TU} := H_{col} + H_{cap} + \frac{H_{brng}}{12}$$

$H_{TU} = 26.53$ feet

E13-1.7 Analyze and Combine Force Effects

The first step within this design step will be to summarize the loads acting on the pier at the bearing locations. This is done in Tables E13-1.7-1 through E13-1.7-8 shown below. Tables E13-1.7-1 through E13-1.7-5 summarize the vertical loads, Tables E13-1.7-6 through E13-1.7-7 summarize the horizontal longitudinal loads, and Table E13-1.7-8 summarizes the horizontal transverse loads. These loads along with the pier self-weight loads, which are

shown after the tables, need to be factored and combined to obtain total design forces to be resisted in the pier cap, column and footing.

It will be noted here that loads applied due to braking and temperature can act either ahead or back station. Also, wind loads can act on either side of the structure and with positive or negative skew angles. This must be kept in mind when considering the signs of the forces in the tables below. The tables assume a particular direction for illustration only.

Bearing	Superstructure Dead Load		Wearing Surface Dead Load	
	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	$Rext_{DC}$	246.65	$Rext_{DW}$	27.32
2	$Rint_{DC}$	241.72	$Rint_{DW}$	27.32
3	$Rint_{DC}$	241.72	$Rint_{DW}$	27.32
4	$Rint_{DC}$	241.72	$Rint_{DW}$	27.32
5	$Rext_{DC}$	246.65	$Rext_{DW}$	27.32

Table E13-1.7-1
Unfactored Vertical Bearing Reactions from Superstructure Dead Load

Bearing	Vehicular Live Load **					
	Lane A		Lane B		Lane C	
	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	R_1_a	4.27	R_1_b	0.00	R_1_c	0.00
2	R_2_a	164.67	R_2_b	0.00	R_2_c	0.00
3	R_3_a	72.31	R_3_b	117.56	R_3_c	0.00
4	R_4_a	0.00	R_4_b	123.66	R_4_c	61.86
5	R_5_a	0.00	R_5_b	0.00	R_5_c	179.40

**Note: Live load reactions include impact on truck loading.

Table E13-1.7-2
Unfactored Vertical Bearing Reactions from Live Load

Strength III

Bearing No.	Reactions from Transverse Wind Load on Superstructure (kips)
1	3.88
2	1.94
3	0.00
4	-1.94
5	-3.88

Strength V

Bearing No.	Reactions from Transverse Wind Load on Superstructure (kips)
1	1.85
2	0.93
3	0.00
4	-0.93
5	-1.85

Service I

Bearing No.	Reactions from Transverse Wind Load on Superstructure (kips)
1	1.41
2	0.70
3	0.00
4	-0.70
5	-1.41

Table E13-1.7-3
Unfactored Vertical Bearing Reactions from Wind on Superstructure

Strength V , Service I

Bearing No.	Reactions from Transverse Wind Load on Vehicular Live Load (kips)
1	2.90
2	1.45
3	0.00
4	-1.45
5	-2.90

Table E13-1.7-4
Unfactored Vertical Bearing Reactions from Wind on Live Load

Strength III

Bearing No.	Variable Name	Reaction (Kips)
	Vertical Wind Load on Superstructure	
1	RWS_{vert1}	4.29
2	RWS_{vert2}	-9.01
3	RWS_{vert3}	-22.32
4	RWS_{vert4}	-35.63
5	RWS_{vert5}	-48.93

<u>Table E13-1.7-5</u>
Unfactored Vertical Bearing Reactions from Vertical Wind on Superstructure

	Braking Load **		Temperature Loading	
Each Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
	BRK_{brg}	3.60	TU_{BRG}	2.79

**Note: Values shown are for a single lane loaded

<u>Table E13-1.7-6</u>
Unfactored Horizontal Longitudinal Bearing Reactions from Braking and Temperature

Strength III

Load Type	Unfactored Horizontal Longitudinal Forces (kips)
Wind Loads from Superstructure	22.34
Wind on Live Load	0.00
Wind on Pier	38.50

Strength V

Load Type	Unfactored Horizontal Longitudinal Forces (kips)
Wind Loads from Superstructure	12.18
Wind on Live Load	9.60
Wind on Pier	18.54

Service I

Load Type	Unfactored Horizontal Longitudinal Forces (kips)
Wind Loads from Superstructure	8.12
Wind on Live Load	9.60
Wind on Pier	14.26

Table E13-1.7-7
Unfactored Horizontal Longitudinal Forces

Strength III

Load Type	Unfactored Horizontal Transverse Forces (kips)
Wind Loads from Superstructure	44.68
Wind on Live Load	0.00
Wind on Pier	5.18

Strength V

Load Type	Unfactored Horizontal Transverse Forces (kips)
Wind Loads from Superstructure	21.32
Wind on Live Load	12.00
Wind on Pier	2.50

Service I

Load Type	Unfactored Horizontal Transverse Forces (kips)
Wind Loads from Superstructure	16.25
Wind on Live Load	12.00
Wind on Pier	1.92

Table E13-1.7-8
Unfactored Horizontal Transverse Forces

In addition to all the loads tabulated above, the pier self-weight must be considered when determining the final design forces. Additionally for the footing and pile designs, the weight of the earth on top of the footing must be considered. These loads were previously calculated and are shown below:

$DL_{Cap} = 251.1$ kips $DL_{ftg} = 144.9$ kips

$DL_{col} = 139.5$ kips $EV_{ftg} = 51.36$ kips

In the AASHTO LRFD design philosophy, the applied loads are factored by statistically calibrated load factors. In addition to these factors, one must be aware of two additional sets of factors which may further modify the applied loads.

The first set of additional factors applies to all force effects and are represented by the Greek letter η (eta) in the Specifications, **LRFD [1.3.2.1]**. These factors are related to the ductility, redundancy, and operational importance of the structure. A single, combined eta is required for every structure. In accordance with WisDOT policy, all eta factors are taken equal to one.

The other set of factors mentioned in the first paragraph above applies only to the live load force effects and are dependent upon the number of loaded lanes. These factors are termed multiple presence factors by the Specifications, **LRFD [Table 3.6.1.1.2-1]**. These factors for this bridge are shown as follows:

Multiple presence factor, m (1 lane) $m_1 := 1.20$

Multiple presence factor, m (2 lanes) $m_2 := 1.00$

Multiple presence factor, m (3 lanes) $m_3 := 0.85$

Table E13-1.7-9 contains the applicable limit states and corresponding load factors that will be used for this pier design. Limit states not shown either do not control the design or are not applicable. The load factors shown in Table E13-1.7-9 are the standard load factors assigned by the Specifications and are exclusive of multiple presence and eta factors.

It is important to note here that the maximum load factors shown in Table E13-1.7-9 for uniform temperature loading (TU) apply only for deformations, and the minimum load factors apply for all other effects. Since the force effects from the uniform temperature loading are considered in this pier design, the minimum load factors will be used.

	Load Factors							
	Strength I		Strength III		Strength V		Service I	
Load	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}
DC	1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.00
DW	1.50	0.65	1.50	0.65	1.50	0.65	1.00	1.00
LL	1.75	1.75	---	---	1.35	1.35	1.00	1.00
BR	1.75	1.75	---	---	1.35	1.35	1.00	1.00
TU	1.20	0.50	1.20	0.50	1.20	0.50	1.20	1.00
WS	---	---	1.00	1.00	1.00	1.00	1.00	1.00
WL	---	---	---	---	1.00	1.00	1.00	1.00
EV	1.35	1.00	1.35	1.00	1.35	1.00	1.00	1.00

Table E13-1.7-9
Load Factors and Applicable Pier Limit States

The loads discussed and tabulated previously can now be factored by the appropriate load factors and combined to determine the governing limit states in the pier cap, column, footing and piles. Design calculations will be carried out for the governing limit states only.

E13-1.7.1 Pier Cap Force Effects

The pier cap will be designed using a strut and tie model. See E13-1.8 for additional information. For this type of model, the member's self weight is included in the bearing reactions. The calculation of the Strength I Factored girder reactions follows.

For the dead load of the cap, the tributary weight of the cap will be added to each girder reaction.

$$Cap_{DC\_1} := 8.625 \cdot \frac{5 + 8.34}{2} \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_1} = 34.52} \qquad \text{kips}$$

$$Cap_{DC\_2} := \left(6.875 \cdot \frac{8.34 + 11}{2} + 2.875 \cdot 11 \right) \cdot W_{cap} \cdot w_c \quad \boxed{Cap_{DC\_2} = 58.86} \qquad \text{kips}$$

$$Cap_{DC\_3} := 9.75 \cdot 11 \cdot W_{cap} \cdot w_c \qquad \boxed{Cap_{DC\_3} = 64.35} \qquad \text{kips}$$

$$Cap_{DC\_4} := Cap_{DC\_2} \qquad \boxed{Cap_{DC\_4} = 58.86} \qquad \text{kips}$$

$$Cap_{DC\_5} := Cap_{DC\_1} \qquad \boxed{Cap_{DC\_5} = 34.52} \qquad \text{kips}$$

Look at the combined live load girder reactions with 1 (Lane C), 2 (Lanes C and B) and 3 lanes (Lanes C, B and A) loaded. The multiple presence factor from E13-1.7 shall be applied. The design lane locations were located to maximize the forces over the right side of the cap.

	Unfactored Vehicular Live Load					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	R_1_1	0.00	R_1_2	0.00	R_1_3	3.63
2	R_2_1	0.00	R_2_2	0.00	R_2_3	139.97
3	R_3_1	0.00	R_3_2	117.56	R_3_3	161.40
4	R_4_1	74.23	R_4_2	185.52	R_4_3	157.70
5	R_5_1	215.27	R_5_2	179.40	R_5_3	152.49

Table E13-1.7-10
Unfactored Vehicular Live Load Reactions

Calculate the Strength I Combined Girder Reactions for 1, 2 and 3 lanes loaded. An example calculation is shown for the girder 5 reaction with one lane loaded. Similar calculations are performed for the remaining girders and number of lanes loaded.

$$Ru_{5\_1} := \gamma_{DCmax} \cdot \left(Rext_{DC} + Cap_{DC\_5} \right) + \gamma_{DWmax} \cdot Rext_{DW} + \gamma_{LL} \cdot R_5\_1$$

$$\boxed{Ru_{5\_1} = 769.17} \qquad \text{kips}$$

Total Factored Girder Reactions**					
1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
Bearing					
1	Ru_1_1 392.44	Ru_1_2	392.44	Ru_1_3	398.79
2	Ru_2_1 416.71	Ru_2_2	416.71	Ru_2_3	661.66
3	Ru_3_1 423.57	Ru_3_2	629.30	Ru_3_3	706.01
4	Ru_4_1 546.62	Ru_4_2	741.38	Ru_4_3	692.68
5	Ru_5_1 769.17	Ru_5_2	706.38	Ru_5_3	659.29

** Includes dead load of pier cap

Table E13-1.7-11
Factored Girder Reactions for STM Cap Design

E13-1.7.2 Pier Column Force Effects

The controlling limit states for the design of the pier column are Strength V (for biaxial bending with axial load). The critical design location is where the column meets the footing, or at the column base. The governing force effects for Strength V are achieved by minimizing the axial effects while maximizing the transverse and longitudinal moments. This is accomplished by excluding the future wearing surface, applying minimum load factors on the structure dead load, and loading only Lane B and Lane C with live load.

For Strength V, the factored vertical forces and corresponding moments at the critical section are shown below.

Strength V Axial Force:

$$Rext_{DC} = 246.65 \quad \text{kips} \qquad R_{3\_2} = 117.56 \quad \text{kips}$$

$$Rint_{DC} = 241.72 \quad \text{kips} \qquad R_{4\_2} = 185.52 \quad \text{kips}$$

$$DL_{Cap} = 251.1 \quad \text{kips} \qquad R_{5\_2} = 179.4 \quad \text{kips}$$

$$DL_{col} = 139.5 \quad \text{kips}$$

$$Ax_{colStrV} := \gamma_{DCminStrV} \cdot \left(2 \cdot Rext_{DC} + 3 \cdot Rint_{DC} + DL_{Cap} + DL_{col} \right) \dots$$
$$+ \gamma_{LLStrV} \left(R_{3\_2} + R_{4\_2} + R_{5\_2} \right)$$

$$Ax_{colStrV} = 2099.51 \quad \text{kips}$$

Strength V Transverse Moment:

$S = 9.75$ feet (girder spacing)

$ArmV3_{col} := 0$ $ArmV3_{col} = 0$ feet

$ArmV4_{col} := S$ $ArmV4_{col} = 9.75$ feet

$ArmV5_{col} := 2 \cdot S$ $ArmV5_{col} = 19.5$ feet

$WS_{suptrnsV} = 21.32$ kips $H_{WStrns} = 30.76$ feet

$WL_{trans} = 12$ kips $H_{WLtrns} = 38.32$ feet

$WS_{subTV} = 2.5$ kips $H_{WSsubT} = 14$ feet

$$MuT_{colStrV} := \gamma_{LLStrV}\left(R_{3\_2} \cdot ArmV3_{col} + R_{4\_2} \cdot ArmV4_{col} + R_{5\_2} \cdot ArmV5_{col}\right) \ldots$$
$$+ \gamma_{WLStrV} \cdot \left(WL_{trans} \cdot H_{WLtrns}\right) \ldots$$
$$+ \gamma_{WSStrV} \cdot \left(WS_{suptrnsV} \cdot H_{WStrns} + WS_{subTV} \cdot H_{WSsubT}\right)$$

$MuT_{colStrV} = 8315.32$ kip-ft

Strength V Longitudinal Moment:

$BRK_{brg} = 3.6$ kips/bearing per lane $H_{BRK} = 26.53$ feet

$TU_{BRG} = 2.79$ kips/ bearing $H_{TU} = 26.53$ feet

$WS_{suplngV} = 12.18$ kips $H_{WSlong} = 26.53$ feet

$WL_{long} = 9.6$ kips $H_{WLlong} = 26.53$ feet

$WS_{subLV} = 18.54$ kips $H_{WSsubL} = 17.11$ feet

$m_2 = 1.00$ multi presence factor for two lanes loaded

$$MuL_{colStrV} := \gamma_{BRStrV} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) \ldots$$
$$+ \gamma_{TUminStrV}\left(5TU_{BRG} \cdot H_{TU}\right) \ldots$$
$$+ \gamma_{WLStrV} \cdot \left(WL_{long} \cdot H_{WLlong}\right) \ldots$$
$$+ \gamma_{WSStrV} \cdot \left(WS_{suplngV} \cdot H_{WSlong} + WS_{subLV} \cdot H_{WSsubL}\right)$$

$MuL_{colStrV} = 2369.38$ kip-ft

For <u>Strength III</u>, the <u>factored transverse shear</u> in the column is:

$$WS_{subTIII} = 5.18 \quad \text{kips} \qquad\qquad WS_{suptrnsIII} = 44.68 \quad \text{kips}$$

$$VuT_{col} := \gamma_{WSStrIII}(WS_{suptrnsIII} + WS_{subTIII}) \qquad VuT_{col} = 49.86 \quad \text{kips}$$

For <u>Strength V</u>, the <u>factored longitudinal shear</u> in the column is (reference Table E13-1.7-7):

$$WL_{long} = 9.6 \quad \text{kips} \qquad WS_{subLV} = 18.54 \quad \text{kips} \qquad WS_{suplngV} = 12.18 \quad \text{kips}$$

$$VuL_{col} := \gamma_{WSStrV}(WS_{suplngV} + WS_{subLV}) + \gamma_{WLStrV} \cdot WL_{long} \ ...$$
$$+ \gamma_{TUmin} \cdot (TU_{BRG} \cdot 5) + \gamma_{BRStrV} \cdot (5 \cdot BRK_{brg}) \cdot 3 \cdot m_3$$

$$VuL_{col} = 109.25 \quad \text{kips}$$

E13-1.7.3 Pier Pile Force Effects

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The force effects in the piles cannot be determined without a pile layout. The pile layout depends upon the pile capacity and affects the footing design. The pile layout used for this pier foundation is shown in Figure E13-1.10-1.

Based on the pile layout shown in Figure E13-1.10-1, the controlling limit states for the pile design are Strength I (for maximum pile load), Strength V (for minimum pile load), and Strength III and Strength V (for maximum horizontal loading of the pile group).

<u>Structure Dead Load Effects</u>:

Girder DC Reactions: Girder DW Reactions:

$$Rext_{DC} = 246.65 \quad \text{kips} \qquad\qquad Rext_{DW} = 27.32 \quad \text{kips}$$

$$Rint_{DC} = 241.72 \quad \text{kips} \qquad\qquad Rint_{DW} = 27.32 \quad \text{kips}$$

$$DC_{Super} := 2 \cdot Rext_{DC} + 3 \cdot Rint_{DC} \qquad DC_{Super} = 1218.46 \quad \text{kips}$$

$$DL_{Cap} = 251.1 \quad \text{kips}$$

$$DL_{col} = 139.5 \quad \text{kips}$$

$$DL_{ftg} = 144.9 \quad \text{kips}$$

$$DC_{pile} := DC_{Super} + DL_{Cap} + DL_{col} + DL_{ftg} \qquad DC_{pile} = 1753.96 \quad \text{kips}$$

$$DW_{pile} := 2 \cdot Rext_{DW} + 3 \cdot Rint_{DW} \qquad DW_{pile} = 136.6 \quad \text{kips}$$

Vertical Earth Load Effects:

$EV_{pile} := EV_{ftg}$　　　　　　　　　　　$EV_{pile} = 51.36$　　　kips

Live Load Effects (without Dynamic Load Allowance)

Live Load Girder Reactions for 2 lanes, Lanes B and C, loaded:

$R_{1\_2p} = 0$　　　　kips
$R_{2\_2p} = 0$　　　　kips
$R_{3\_2p} = 99.21$　　kips
$R_{4\_2p} = 156.54$　kips
$R_{5\_2p} = 151.38$　kips　　　　　　　　$R_{T\_2p} = 407.13$　kips

From Section E13-1.7, the Transverse moment arm for girders 3, 4 and 5 are:

$ArmV3_{col} = 0$　　feet

$ArmV4_{col} = 9.75$　feet

$ArmV5_{col} = 19.5$　feet

The resulting Transverse moment applied to the piles is:

$M_{LL2T\_p} := R_{3\_2p} \cdot ArmV3_{col} + R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}$

$M_{LL2T\_p} = 4478.2$　kip-ft

The Longitudinal Strength I Moment includes the braking and temperature forces.

$MuL2_{colStr1} := \gamma_{BR} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) + \gamma_{TUmin}\left(5TU_{BRG} \cdot H_{TU}\right)$

$MuL2_{colStr1} = 1856.29$　kip-ft

Strength I Load for Maximum Pile Reaction

The maximum pile load results from the Strength I load combination with two lanes loaded.

$Pu2_{pile\_Str1} := \gamma_{DCmax} \cdot DC_{pile} + \gamma_{DWmax} \cdot DW_{pile} + \gamma_{EVmax} \cdot EV_{pile} + \gamma_{LL} \cdot R_{T\_2p}$

$$\boxed{Pu2_{pile\_Str1} = 3179.17} \quad \text{kips}$$

$$MuT2_{pile\_Str1} := \gamma_{LL} \cdot M_{LL2T\_p} \qquad \boxed{MuT2_{pile\_Str1} = 7836.85} \quad \text{kip-ft}$$

$$MuL2_{pile\_Str1} := MuL2_{colStr1} \qquad \boxed{MuL2_{pile\_Str1} = 1856.29} \quad \text{kip-ft}$$

Minimum Load on Piles Strength V

The calculation for the minimum axial load on piles is similar to the Strength V axial column load calculated previously. The weight of the footing and soil surcharge are included. The girder reactions used for pile design do not include impact. The DW loads are not included.

$$Pu_{pile\_StrV} := \gamma_{DCminStrV} \cdot \left(2 \cdot Rext_{DC} + 3 \cdot Rint_{DC} + DL_{Cap} + DL_{col} + DL_{ftg}\right) \ldots$$
$$+ \gamma_{EVminStrV} \cdot EV_{pile} \ldots$$
$$+ \gamma_{LLStrV}\left(R_{3\_2p} + R_{4\_2p} + R_{5\_2p}\right)$$

$$\boxed{Pu_{pile\_StrV} = 2179.55} \quad \text{kips}$$

The calculation for the Strength V longitudinal moment is the same as the longitudinal moment on the column calculated previously. These loads include the braking force, temperature, wind on live load and wind on the structure.

$$MuL_{pile\_StrV} := \gamma_{BRStrV} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) \ldots$$
$$+ \gamma_{TUminStrV}\left(5TU_{BRG} \cdot H_{TU}\right) \ldots$$
$$+ \gamma_{WLStrV} \cdot \left(WL_{long} \cdot H_{WLlong}\right) \ldots$$
$$+ \gamma_{WSStrV} \cdot \left(WS_{suplngV} \cdot H_{WSlong} + WS_{subLV} \cdot H_{WSsubL}\right)$$

$$\boxed{MuL_{pile\_StrV} = 2369.38} \quad \text{kip-ft}$$

The calculation for the Strength V transverse moment is the similar as the transverse moment on the column calculated previously. These loads include the live load, wind on live load and wind on the structure. Impact is not included in these live load reactions.

$$MuT_{pile\_StrV} := \gamma_{LLStrV}\left(R_{3\_2p} \cdot ArmV3_{col} + R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col}\right) \ldots$$
$$+ \gamma_{WLStrV} \cdot \left(WL_{trans} \cdot H_{WLtrns}\right) \ldots$$
$$+ \gamma_{WSStrV} \cdot \left(WS_{suptrnsV} \cdot H_{WStrns} + WS_{subTV} \cdot H_{WSsubT}\right)$$

$$\boxed{MuT_{pile\_StrV} = 7196.34} \quad \text{kip-ft}$$

For <u>Strength III</u>, the <u>factored transverse shear</u> in the footing is equal to the transverse force at the base of the column.

$$HuT_{pileStrIII} := VuT_{col}$$

$$= \gamma_{WSStrIII} \cdot (WS_{suptrnsIII} + WS_{subTIII}) \qquad \boxed{HuT_{pileStrIII} = 49.86} \quad kips$$

For <u>Strength V</u>, the <u>factored longitudinal shear</u> in the column is equal to the longitudinal force at the base of the column.

$$HuL_{pileStrV} := VuL_{col} \qquad\qquad\qquad \boxed{HuL_{pileStrV} = 109.25} \quad kips$$

The following is a <u>summary</u> of the controlling forces on the piles:

<u>Strength I</u>

$\boxed{Pu2_{pile\_Str1} = 3179.17}$ kips

$\boxed{MuT2_{pile\_Str1} = 7836.85}$ kip-ft

$\boxed{MuL2_{pile\_Str1} = 1856.29}$ kip-ft

<u>Strength III</u>

$\boxed{HuT_{pileStrIII} = 49.86}$ kips

<u>Strength V</u>

$\boxed{Pu_{pile\_StrV} = 2179.55}$ kips

$\boxed{MuT_{pile\_StrV} = 7196.34}$ kip-ft

$\boxed{MuL_{pile\_StrV} = 2369.38}$ kip-ft

$\boxed{HuL_{pileStrV} = 109.25}$ kips

E13-1.7.4 Pier Footing Force Effects

The controlling limit states for the design of the pier footing are <u>Strength I</u> (for <u>flexure</u>, <u>punching shear</u> at the <u>column</u>, and <u>punching shear</u> at the <u>maximum loaded pile</u>, and for <u>one-way shear</u>). In accordance with Section 13.11, the footings do not require the crack control by distribution check in **LRFD [5.6.7]**. As a result, the Service I Limit State is not required. There is not a single critical design location in the footing where all of the force effects just mentioned are checked. Rather, the force effects act at different locations in the footing and must be checked at their respective locations. For example, the punching shear checks are carried out using critical perimeters around the column and maximum loaded pile,

while the flexure and one-way shear checks are carried out on a vertical face of the footing either parallel or perpendicular to the bridge longitudinal axis. Also note that impact is not included for members that are below ground. The weight of the footing concrete and the soil above the footing are not included in these loads as they counteract the pile reactions.

$$DC_{ftg} := DC_{Super} + DL_{Cap} + DL_{col} \qquad \boxed{DC_{ftg} = 1609.06} \quad \text{kips}$$

$$DW_{ftg} := 2 \cdot Rext_{DW} + 3 \cdot Rint_{DW} \qquad \boxed{DW_{ftg} = 136.6} \quad \text{kips}$$

Unfactored Live Load reactions for <u>one</u>, <u>two</u> and <u>three lanes loaded</u>:

$$\boxed{R_{T\_1p} = 244.3} \quad \text{kips}$$

$$\boxed{R_{T\_2p} = 407.13} \quad \text{kips}$$

$$\boxed{R_{T\_3p} = 519.1} \quad \text{kips}$$

The resulting <u>Transverse moment</u> applied to the piles is:

$$M_{LL1T} := R_{4\_1p} \cdot ArmV4_{col} + R_{5\_1p} \cdot ArmV5_{col} \qquad \boxed{M_{LL1T} = 4153.03} \quad \text{kip-ft}$$

$$M_{LL2T} := R_{4\_2p} \cdot ArmV4_{col} + R_{5\_2p} \cdot ArmV5_{col} \qquad \boxed{M_{LL2T} = 4478.2} \quad \text{kip-ft}$$

$$M_{LL3T} := \left(-R_{2\_3p} + R_{4\_3p}\right) \cdot ArmV4_{col} + \left(-R_{1\_3p} + R_{5\_3p}\right) \cdot ArmV5_{col}$$

$$\boxed{M_{LL3T} = 2595.17} \quad \text{kip-ft}$$

The <u>maximum pile load</u> results from the <u>Strength I</u> load combination with <u>two lanes loaded</u>.

$$Pu2_{ftgStr1} := \gamma_{DCmax} \cdot DC_{ftg} + \gamma_{DWmax} \cdot DW_{ftg} + \gamma_{LL} \cdot R_{T\_2p}$$

$$\boxed{Pu2_{ftgStr1} = 2928.7} \quad \text{kips}$$

$$MuT2_{ftgStr1} := \gamma_{LL} \cdot M_{LL2T} \qquad \boxed{MuT2_{ftgStr1} = 7836.85} \quad \text{kip-ft}$$

$$MuL2_{ftgStr1} := \gamma_{BR} \cdot \left(5 \cdot BRK_{brg} \cdot H_{BRK} \cdot 2 \cdot m_2\right) \dots \qquad \boxed{MuL2_{ftgStr1} = 1856.29} \quad \text{kip-ft}$$
$$\qquad\qquad + \gamma_{TUmin}\left(5TU_{BRG} \cdot H_{TU}\right)$$

The <u>Strength I</u> limit state controls for the <u>punching shear check</u> at the column. In this case the future wearing surface is included, maximum factors are applied to all the dead load components, and all <u>three lanes are loaded</u> with live load. This results in the following bottom of column forces:

$$Pu3_{ftgStr1} := \gamma_{DCmax} \cdot DC_{ftg} + \gamma_{DWmax} \cdot DW_{ftg} + \gamma_{LL} \cdot R_{T\_3p}$$

$$\boxed{Pu3_{ftgStr1} = 3124.66} \quad \text{kips}$$

$$\text{MuT3}_{ftgStr1} := \gamma_{LL} \cdot M_{LL3T}$$

$$\boxed{\text{MuT3}_{ftgStr1} = 4541.55} \quad \text{kip-ft}$$

$$\text{MuL3}_{ftgStr1} := \gamma_{BR} \cdot \left(5 \cdot \text{BRK}_{brg} \cdot H_{BRK} \cdot 3 \cdot m_3\right) \ldots$$
$$+ \gamma_{TUmin}\left(5TU_{BRG} \cdot H_{TU}\right)$$

$$\boxed{\text{MuL3}_{ftgStr1} = 2315.94} \quad \text{kip-ft}$$

E13-1.8 Design Pier Cap - Strut and Tie Method (STM)

Prior to carrying out the actual design of the pier cap, a brief discussion is in order regarding the design philosophy that will be used for the design of the structural components of this pier.

When a structural member meets the definition of a deep component **LRFD [5.8.2.1]**, the LRFD Specifications recommend, although it does not mandate, that the strut-and-tie method be used to determine force effects and required reinforcing. **LRFD [C5.8.2.1]** indicates that a strut-and-tie model properly accounts for nonlinear strain distribution, nonuniform shear distribution, and the mechanical interaction of V_u, T_u and M_u. Use of strut-and-tie models for the design of reinforced concrete members is new to the LRFD Specification. <u>WisDOT policy is to design hammerhead pier caps using STM</u>.

E13-1.8.1 Determine Geometry and Member Forces
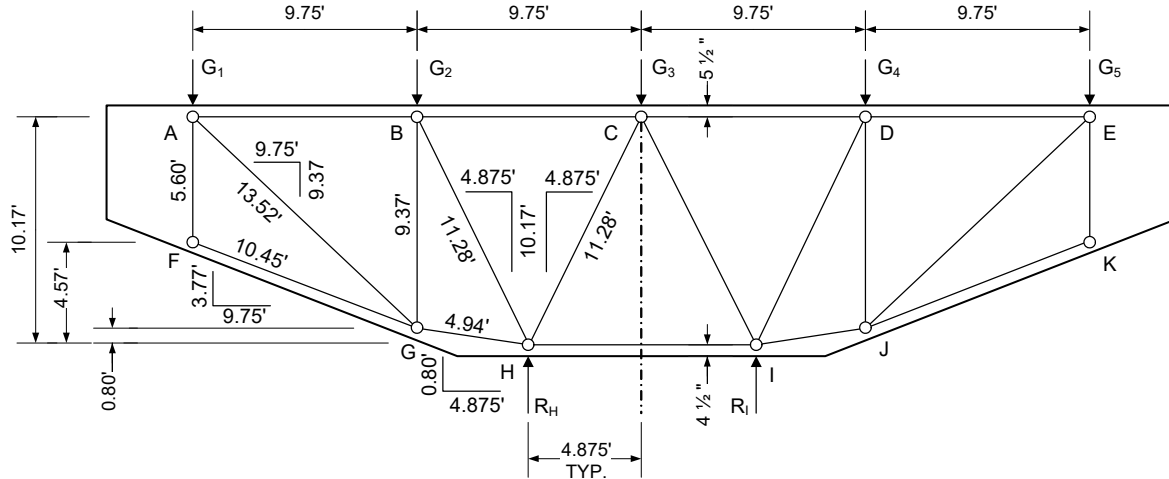
<u>Figure E13-1.8-1</u>
Strut and Tie Model Dimensions

In order to maintain a minimum 25° angle between struts and ties, the support <u>Nodes</u> (<u>H</u> and <u>I</u>) are located midway between the girder reactions **LRFD [5.8.2.2]**. For this example a compressive strut depth of 8 inches will be used, making the centroids of the bottom truss chords 4.5 inches from the concrete surface. It is also assumed that two layers of rebar will be required along the top tension ties, and the centroid is located 5.5 inches below the top of the cap.

$$\text{centroid}_{bot} := 4.5 \quad \text{inches} \qquad \text{centroid}_{top} := 5.5 \quad \text{inches}$$

Strength I Loads:

	Total Factored Girder Reactions**					
	1 Lane, m=1.2		2 Lanes, m=1.0		3 Lanes, m=0.85	
Bearing	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)	Variable Name	Reaction (Kips)
1	Ru_1_1	392.44	Ru_1_2	392.44	Ru_1_3	398.79
2	Ru_2_1	416.71	Ru_2_2	416.71	Ru_2_3	661.66
3	Ru_3_1	423.57	Ru_3_2	629.30	Ru_3_3	706.01
4	Ru_4_1	546.62	Ru_4_2	741.38	Ru_4_3	692.68
5	Ru_5_1	769.17	Ru_5_2	706.38	Ru_5_3	659.29

** Includes dead load of pier cap

Table E13-1.8-1
Total Factored Girder Reactions

Calculate the forces in the members for the Strength I Load Case with 2 lanes loaded.

To find the column reaction at Node I, sum moments about Node H:

$$R_{I\_2} := \frac{Ru_{3\_2} \cdot 4.875 + Ru_{4\_2} \cdot 14.625 + Ru_{5\_2} \cdot 24.375 - Ru_{2\_2} \cdot 4.875 - Ru_{1\_2} \cdot 14.625}{9.75}$$

$\boxed{R_{I\_2} = 2395.66}$ kips

$$R_{H\_2} := Ru_{1\_2} + Ru_{2\_2} + Ru_{3\_2} + Ru_{4\_2} + Ru_{5\_2} - R_{I\_2}$$

$\boxed{R_{H\_2} = 490.55}$ kips

The method of joints is used to calculate the member forces. Start at Node K.

By inspection, the following are zero force members and can be ignored in the model:

$F_{JK} := 0$ $F_{EK} := 0$ $F_{AF} := 0$ $F_{FG} := 0$ kips

Note: All forces shown are in kips. "C" indicates compression and "T" indicates tension.

At Node E:

$F_{EJ\_vert} := Ru_{5\_2}$ $\boxed{F_{EJ\_vert} = 706.38}$

$$F_{EJ\_horiz} = Ru_{5\_2} \cdot \frac{EJ_h}{EJ_v}$$

$$\boxed{F_{EJ\_horiz} = 735.42}$$

$$F_{EJ} := \sqrt{F_{EJ\_vert}^2 + F_{EJ\_horiz}^2}$$

$$\boxed{F_{EJ} = 1019.71} \quad C$$

$$F_{DE} := F_{EJ\_horiz}$$

$$\boxed{F_{DE} = 735.42} \quad T$$

At <u>Node J</u>:

$$F_{IJ\_horiz} := F_{EJ\_horiz}$$

$$\boxed{F_{IJ\_horiz} = 735.42}$$

$$F_{IJ\_vert} = F_{IJ\_horiz} \cdot \frac{0.802}{4.875}$$

$$\boxed{F_{IJ\_vert} = 120.99}$$

$$F_{IJ} := \sqrt{F_{IJ\_horiz}^2 + F_{IJ\_vert}^2}$$

$$\boxed{F_{IJ} = 745.31} \quad C$$

$$F_{DJ} := F_{EJ\_vert} - F_{IJ\_vert}$$

$$\boxed{F_{DJ} = 585.4} \quad T$$

At <u>Node D</u>:

$$F_{DI\_vert} := F_{DJ} + Ru_{4\_2}$$

$$\boxed{F_{DI\_vert} = 1326.77}$$

$$F_{DI\_horiz} = F_{DI\_vert} \cdot \frac{4.875}{10.167}$$

$$\boxed{F_{DI\_horiz} = 636.18}$$

$$F_{DI} := \sqrt{F_{DI\_vert}^2 + F_{DI\_horiz}^2}$$

$$\boxed{F_{DI} = 1471.41} \quad C$$

$$F_{CD} := F_{DE} + F_{DI\_horiz}$$

$$\boxed{F_{CD} = 1371.6} \quad T$$

At <u>Node I</u>:

$$\boxed{R_{I\_2} = 2395.66}$$

$$F_{CI\_vert} := R_{I\_2} - F_{DI\_vert} - F_{IJ\_vert}$$

$$\boxed{F_{CI\_vert} = 947.9}$$

$$F_{CI\_horiz} = F_{CI\_vert} \cdot \frac{4.875}{10.167}$$

$$\boxed{F_{CI\_horiz} = 454.51}$$

$$F_{CI} := \sqrt{F_{CI\_vert}^2 + F_{CI\_horiz}^2}$$

$$\boxed{F_{CI} = 1051.23} \quad C$$

$$F_{HI} := F_{DI\_horiz} + F_{IJ\_horiz} - F_{CI\_horiz}$$

$$\boxed{F_{HI} = 917.09} \quad C$$

Similar calculations are performed to determine the member forces for the remainder of the model and for the load cases with <u>one and three lanes loaded</u>. The results are summarized in the following figures:
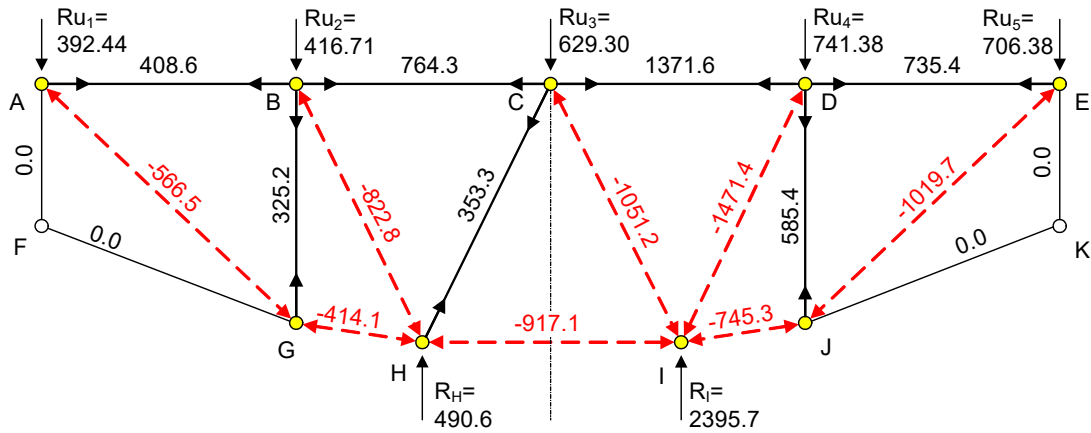
Figure E13-1.8-2
STM Member Forces (Two Lanes Loaded)
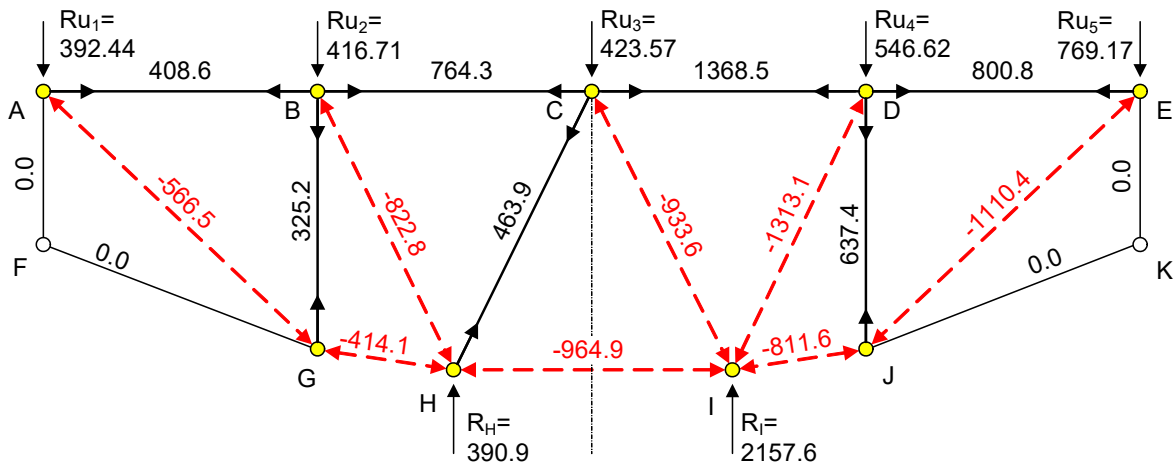
Figure E13-1.8-3
STM Member Forces (One Lane Loaded)
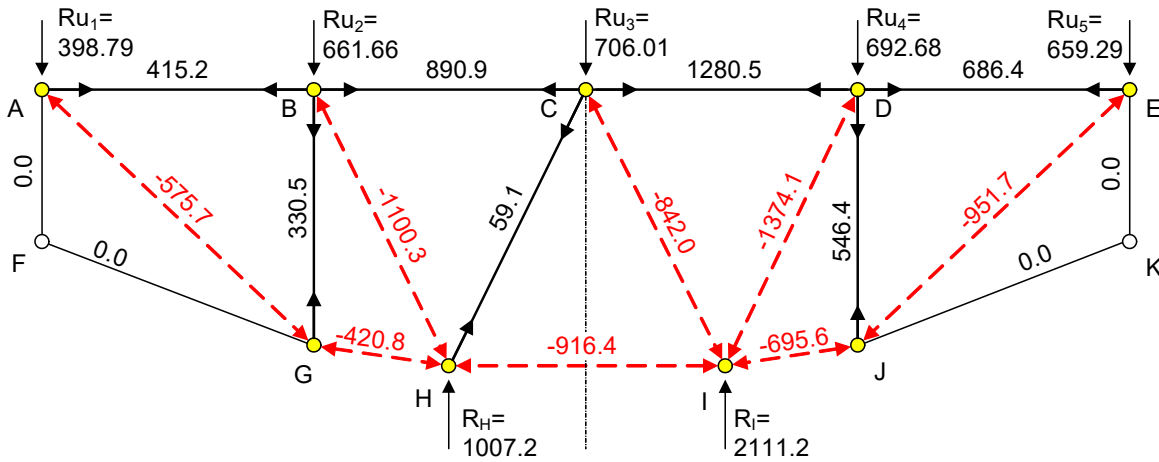
Figure E13-1.8-4
STM Member Forces (Three Lanes Loaded)

E13-1.8.2 Check the Size of the Bearings

The node types are defined by the combinations of struts and ties meeting at the node.

Nodes may be characterized as:

- CCC: Nodes where only struts intersect

- CCT: Nodes where a tie intersects the node in only one direction

- CTT: Nodes where ties intersect in two different directions

The nominal resistance (P_n) at the bearing node face is computed based on the limiting compressive stress (f_{cu}), and the effective area beneath the bearing device ($A_{bearing}$) **LRFD [5.8.2.5]**.

$$P_n = f_{cu} A_{bearing} = (m \, \nu \, f'_c) A_{bearing}$$

where:

m = Confinement modification factor **LRFD [5.6.5]**

ν = Concrete efficiency factor **LRFD [5.8.2.5.3a]**

therefore, $A_{bearing} \geq P_u / \phi_{brg} (m \, \nu \, f'_c)$

Bearing Nodes: Nodes A & E: (CCT)   Node C: (CTT)   Nodes B & D: (CTT)

The nodes located at the bearings are either (CTT) or (CCT) nodes, and the largest loads for these types are present at Nodes D and E respectively. Conservatively use m=1.0, and analyze for crack control reinforcement being present.

At Node D the bearing area required is: (CTT)

$$A_{bearing} \geq P_u / \phi_{brg}\left(m \cdot 0.65 \cdot f'_c\right) \quad \text{---} \quad \text{(from Sect. 13.7.3)}$$

At Node E the bearing area required is: (CCT)

$$A_{bearing} \geq P_u / \phi_{brg}\left(m \cdot 0.70 \cdot f'_c\right) \quad \text{---} \quad \text{(from Sect. 13.7.3)}$$

$m := 1.0$ $\phi_{brg} := 0.70$ **LRFD [5.5.4.2]** $f'_c = 3.5$ ksi

Calculate bearing area required for Node D:

$\boxed{Ru_{4\_2} = 741.38}$ kips   2-lanes loaded controls (Fig. E13-1.8-2)

$\boxed{\gamma_{DCmax} \cdot Cap_{DC\_4} = 73.58}$ kips   pier cap tributary weight below Node D

$$BrgD_2 := \frac{Ru_{4\_2} - \gamma_{DCmax} \cdot Cap_{DC\_4}}{\phi_{brg} \cdot \left(m \cdot 0.65 \cdot f'_c\right)} \qquad \boxed{BrgD_2 = 419.34} \qquad in^2$$

Calculate bearing area required for Node E:

$\boxed{Ru_{5\_1} = 769.17}$ kips   1-lane loaded controls (Fig. E13-1.8-3)

$\boxed{\gamma_{DCmax} \cdot Cap_{DC\_5} = 43.15}$ kips   pier cap tributary weight below Node E

$$BrgE_1 := \frac{Ru_{5\_1} - \gamma_{DCmax} \cdot Cap_{DC\_5}}{\phi_{brg} \cdot \left(m \cdot 0.70 \cdot f'_c\right)} \qquad \boxed{BrgE_1 = 423.34} \qquad in^2$$

$$BrgArea := max\left(BrgD_2, BrgE_1\right)$$

The area provided by the (26" x 18") bearing plate is:

$$A_{bearing} := L_{brng} \cdot W_{brng} \qquad \boxed{A_{bearing} = 468} \qquad in^2$$

Is $A_{bearing} \geq BrgArea$? $\boxed{check = \text{"OK"}}$

E13-1.8.3 Calculate the Tension Tie Reinforcement

For the top reinforcement in the pier cap, the maximum area of tension tie reinforcement, (A_{st}), is controlled by Tie CD for two lanes loaded (Fig. E13-1.8-2) and is calculated as follows:

LRFD [5.8.2.4.1]

$Pu_{CD\_2} = 1371.6$ kips

$\phi := 0.9$ **LRFD [5.5.4.2]** $f_y = 60$ ksi

$Ast_{CD} := \dfrac{Pu_{CD\_2}}{\phi \cdot f_y}$ $Ast_{CD} = 25.4$ in²

Therefore, use one row of 9 No.11 bars and one row of 9 No. 10 bars spaced at 5 inches for the top reinforcement.

$As_{No11} := 1.5625$ in²

$As_{No10} := 1.2656$ in²

Total area of top reinforcement is:

$As_{CD} := 9 \cdot As_{No11} + 9 \cdot As_{No10}$ $As_{CD} = 25.45$ in²

Is $As_{CD} \geq Ast_{CD}$?   check = "OK"
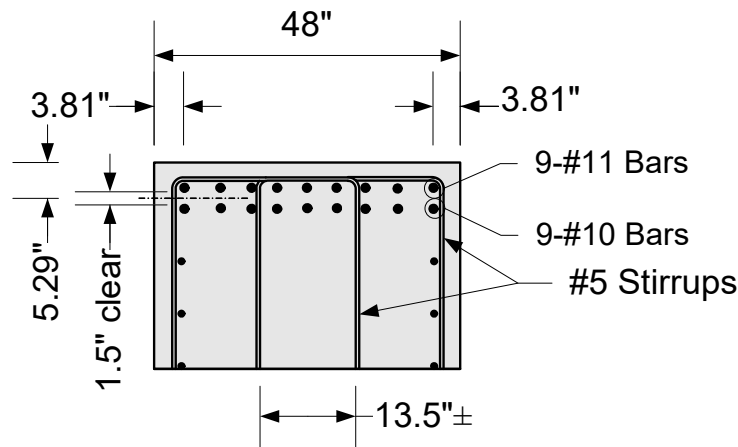
Figure E13-1.8-5
Cap Reinforcement at Tension Tie CD

Note: See **LRFD [5.10.3.1.3]** for spacing requirements between layers of rebar.

For the top reinforcement just inside the exterior girder (Node E), the required area of tension tie reinforcement, (A_{st}), is controlled by Tie DE for one lane loaded (Fig. E13-1.8-3), and is calculated as follows:

$Pu_{DE\_1} = 800.79$ kips

$\phi = 0.9$ **LRFD [5.5.4.2]** $f_y = 60$ ksi

$Ast_{DE} := \dfrac{Pu_{DE\_1}}{\phi \cdot f_y}$ $\boxed{Ast_{DE} = 14.83}$ in^2

Therefore, use one row of 9 No.11 bars spaced at 5 inches, and one row of 5 No.10 bars for the top reinforcement.

Total area of top reinforcement is:

$As_{DE} := 9 \cdot As_{No11} + 5 \cdot As_{No10}$ $\boxed{As_{DE} = 20.39}$ in^2

Is $As_{DE} \geq Ast_{DE}$? $\boxed{check = \text{"OK"}}$
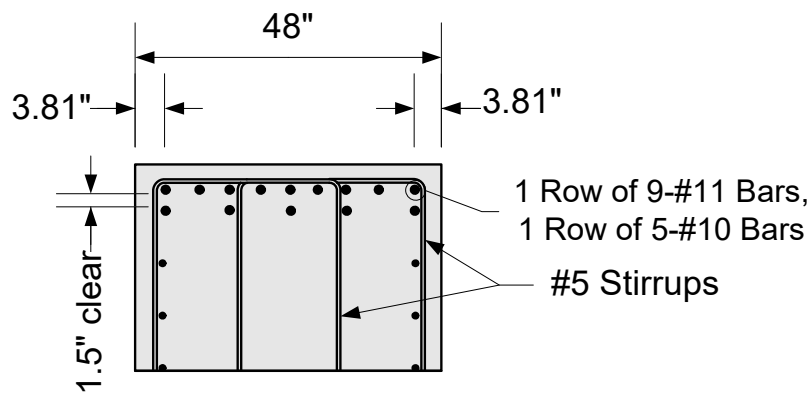
Figure E13-1.8-6
Cap Reinforcement at Tension Tie DE

E13-1.8.4 Calculate the Stirrup Reinforcement

The vertical tension <u>Tie DJ</u> must resist a factored tension force as shown below **LRFD [5.8.2.4.1]**. The controlling force occurs with one lane loaded (Fig. E13-1.8-3). This tension force will be resisted by stirrups within the specified stirrup region length, with the total area of stirrups being (Ast_{DJ}.) Note that any tension ties located directly over the column do not require stirrup design.

$\boxed{Pu_{DJ\_1} = 637.43}$ kips

$\phi = 0.9$ **LRFD [5.5.4.2]** $f_y = 60$ ksi

$Ast_{DJ} := \dfrac{Pu_{DJ\_1}}{\phi \cdot f_y}$ $\boxed{Ast_{DJ} = 11.8}$ in^2

Try No. 5 bars, with four legs (double-stirrups):

$As_{No5} := 0.3068$ in^2

$Ast := 4 \cdot As_{No5}$ $\boxed{Ast = 1.23}$ in^2

Calculate number of stirrups required:

$n_{DJ} := \dfrac{Ast_{DJ}}{Ast}$ $n_{DJ} = 9.62$

$\boxed{n_{DJ} = 10}$ bars

The length (L_{DJ}) of the region over which the stirrups shall be distributed for <u>Tie DJ</u>, is from the face of the column to half way between girders 4 and 5 (<u>Nodes D and E</u>).

$\boxed{S = 9.75}$ feet (girder spacing) $\boxed{L_{col} = 15.5}$ feet (column width)

$L_{DJ} := 1.5 \cdot S - \dfrac{L_{col}}{2}$ $\boxed{L_{DJ} = 6.88}$ feet

Therefore, the required stirrup spacing, s, within this region is:

$s_{stirrup} := \dfrac{L_{DJ} \cdot 12}{n_{DJ}}$ $\boxed{s_{stirrup} = 8.25}$ in

$\boxed{s_{stirrup} = 8}$ in

Examine stirrups as vertical crack control reinforcement, and their req'd. spacing (s_{cc})
LRFD [5.8.2.6]:

$\dfrac{Ast}{b_v \cdot s_{cc}} \geq 0.003$

$b_v := W_{cap} \cdot 12$ $\boxed{b_v = 48}$ in

$s_{cc} := \dfrac{Ast}{0.003 \cdot b_v}$ $\boxed{s_{cc} = 8.52}$ in

$\boxed{s_{cc} = 8}$ in

$s_{stir} := min\left(s_{stirrup}, s_{cc}\right)$ $\boxed{s_{stir} = 8}$ in

Therefore, use pairs of (No. 5 bar) double-legged stirrups at 8 inch spacing in the pier cap.

E13-1.8.5 Compression Strut Capacity - Bottom Strut

After the tension tie reinforcement has been designed, the next step is to check the capacity of the compressive struts in the pier cap. <u>Strut IJ</u> carries the highest bottom compressive force when one lane is loaded (Fig. E13-1.8-3). <u>Strut IJ</u> is anchored by <u>Node J</u>, which also anchors <u>Tie DJ</u> and <u>Strut EJ.</u> From the geometry of the idealized internal truss, the smallest angle (α_s) between <u>Tie DJ</u> and <u>Strut IJ</u> is:

$$\alpha_s := atan\left(\frac{IJ_h}{IJ_v}\right) \qquad\qquad \boxed{\alpha_s = 80.66 \cdot deg}$$

$$\theta := 90deg - \alpha_s \qquad\qquad \boxed{\theta = 9.34 \cdot deg}$$

$$\boxed{Pu_{IJ\_1} = -811.55} \quad kips$$

The nominal resistance (Pn_{IJ}) of <u>Strut IJ</u> is computed based on the limiting compressive stress, (f_{cu}), and the effective cross-sectional area of the strut (Acn_{IJ}) at the node face **LRFD [5.8.2.5]**.

$$Pn_{IJ} = f_{cu} \, Acn_{IJ} = (v \, f_c) \, Acn_{IJ}$$

where:

v = Concrete efficiency factor **LRFD [5.8.2.5.3a]**

therefore, $Pu_{IJ\_1} \le \phi c_{STM} (v \, f_c) \, Acn_{IJ}$

The centroid of the strut was assumed to be at $\boxed{centroid_{bot} = 4.5}$ inches vertically from the bottom face. Therefore at <u>Node J</u>, the thickness of the strut perpendicular to the sloping bottom face (t_{IJ}), and the width (w_{IJ}) of the strut are:

$$t_{IJ} := 2 \cdot centroid_{bot} \cdot cos(\theta) \qquad\qquad \boxed{t_{IJ} = 8.88} \qquad inches$$

$$w_{IJ} := W_{cap} \cdot 12 \qquad\qquad \boxed{w_{IJ} = 48} \qquad inches$$

$$Acn_{IJ} := t_{IJ} \cdot w_{IJ} \qquad\qquad \boxed{Acn_{IJ} = 426.27} \qquad in^2$$

At <u>Node J</u> the node type is (CCT), and the surface where <u>Strut IJ</u> meets the node is a <u>back face</u>. Analyze for crack control reinforcement being present.

At <u>Node J</u>, the capacity of <u>Strut IJ</u> shall satisfy:

$$Pu_{IJ\_1} \le \phi c_{STM} \cdot (0.70 \cdot f_c) \cdot Acn_{IJ} \quad \text{---} \quad (from\ Sect.\ 13.7.5)$$

$\phi c_{STM} := 0.7$ **LRFD [5.5.4.2]** $f_c = 3.5$ ksi

<u>The factored resistance is:</u>

$$Pr_{IJ} := \phi c_{STM} \cdot (0.70 \cdot f'_c) \cdot Acn_{IJ}$$

$$Pr_{IJ} = 731.05 \quad \text{kips}$$

$$\left| Pu_{IJ\_1} \right| = 811.55 \quad \text{kips}$$

Is $Pr_{IJ} \geq Pu_{IJ\_1}$?

check = "No Good"

Because Node J is an interior node not bounded by a bearing plate, it is a smeared node, and a check of concrete strength as shown above is not necessary **LRFD [5.8.2.2]**.

E13-1.8.6 Compression Strut Capacity - Diagonal Strut

Strut DI carries the highest diagonal compressive force when two lanes are loaded (Fig. E13-1.8-2). Strut DI is anchored by Node D, which also anchors Ties CD, DE and DJ. From the geometry of the idealized internal truss, the smallest angle between Ties CD and DE and Strut DI is:

$$\alpha_s := atan\left(\frac{DI_v}{DI_h}\right)$$

$$\alpha_s = 64.38 \cdot deg$$

$$\theta := 90deg - \alpha_s$$

$$\theta = 25.62 \cdot deg$$

$$Pu_{DI\_2} = -1471.41 \quad \text{kips}$$

The cross sectional dimension of Strut DI in the plane of the pier at Node D is calculated as follows. Note that for skewed bearings, the length of the bearing is the projected length along the centerline of the pier cap.

$$L_{brng} = 26 \quad \text{inches} \qquad W_{brng} = 18 \quad \text{inches}$$

$$centroid_{top} = 5.5 \quad \text{inches}$$
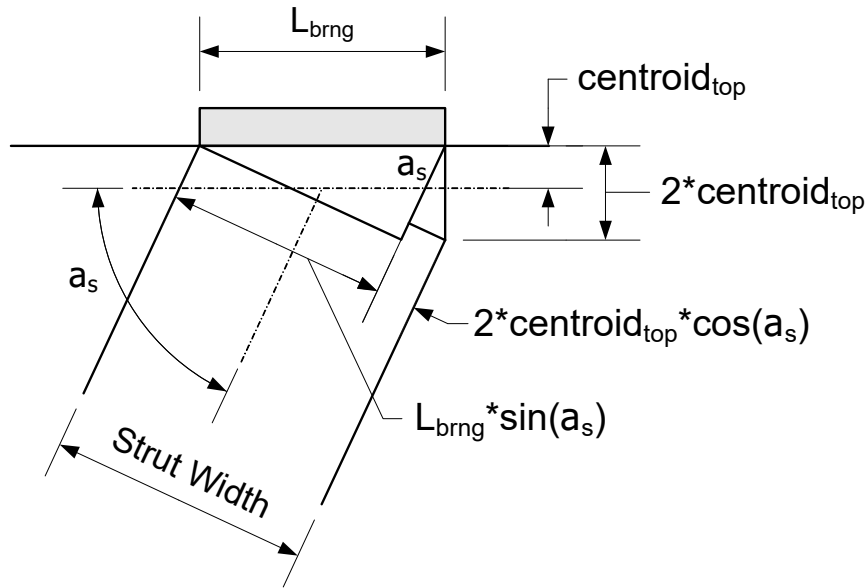
Figure E13-1.8-7
Compression Strut Width

Therefore at <u>Node D</u>, the thickness of the strut (t_{DI}) is:

$$t_{DI} := L_{brng} \cdot \sin(\alpha_s) + 2 \cdot centroid_{top} \cdot \cos(\alpha_s)$$ $\boxed{t_{DI} = 28.2}$ in

The effective compression strut width around each stirrup is:

$d_{bar11} := 1.410$ inches

$$w_{ef} := 2 \cdot 6 \cdot d_{bar11}$$ $\boxed{w_{ef} = 16.92}$ in

The effective spacing between the 4 legs of the stirrups is 13.5 inches, which is less than the value calculated above. Therefore, the entire cap width can be used for the effective strut width.

$$w_{DI} := W_{cap} \cdot 12$$ $\boxed{w_{DI} = 48}$ in

The nominal resistance (Pn_{DI}) of <u>Strut DI</u> is computed based on the limiting compressive stress, (f_{cu}), and the effective cross-section of the strut (Acn_{DI}) at the node face **LRFD [5.8.2.5]**.

$$Acn_{DI} := t_{DI} \cdot w_{DI}$$ $\boxed{Acn_{DI} = 1353.61}$ in²

At <u>Node D</u> the node type is (CTT), and the surface where <u>Strut DI</u> meets the node is a <u>strut to node interface</u>. Analyze for crack control reinforcement being present.

At <u>Node D</u>, the capacity of <u>Strut DI</u> shall satisfy:

$Pu_{DI\_2} \leq \phi c_{STM} \cdot (0.65 \cdot f'_c) \cdot Acn_{DI}$ —   (from Sect. 13.7.5)

$\phi c_{STM} := 0.7$ **LRFD [5.5.4.2]** $f'_c = 3.5$ ksi

The factored resistance is:

$Pr_{DI} := \phi c_{STM} \cdot (0.65 \cdot f'_c) \cdot Acn_{DI}$ $\boxed{Pr_{DI} = 2155.62}$ kips

                                                $\boxed{\left| Pu_{DI\_2} \right| = 1471.41}$ kips

Is $Pr_{DI} \geq \left| Pu_{DI\_2} \right|$? $\boxed{\text{check} = \text{"OK"}}$

E13-1.8.7 Check the Anchorage of the Tension Ties

Tension ties shall be anchored in the nodal regions per **LRFD [5.8.2.4.2]**. The 9 No. 11 and 5 No. 10 longitudinal bars along the top of the pier cap must be developed at the inner edge of the bearing at Node E (the edge furthest from the end of the member). Based on (Figure E13-1.8-8), the embedment length that is _available_ to develop the bar beyond the edge of the bearing is:

L_{devel} = (distance from cap end to Node E) + (bearing block width/2) - (cover)

$\boxed{L_{cap} = 46.5}$ feet   (pier cap length)

$\boxed{S = 9.75}$ feet   (girder spacing) $\boxed{ng = 5}$ (# girders)

$\boxed{L_{brng} = 26}$ inches   (bearing block width)

$\boxed{Cover_{cp} = 2.5}$ inches   (conc. cover)

$$L_{devel} := \frac{L_{cap} - S \cdot (ng - 1)}{2} \cdot 12 + \frac{L_{brng}}{2} - Cover_{cp} \quad \boxed{L_{devel} = 55.5} \quad \text{in}$$

The basic development length for straight No. 11 and No. 10 bars with spacing less than 6", As(provided)/As(required) < 2, uncoated top bar, per (Wis Bridge Manual Table 9.9-1) is:

$\boxed{L_{d11} := 9.5}$ ft $\boxed{L_{d11} \cdot 12 = 114}$ in

$\boxed{L_{d10} := 7.75}$ ft $\boxed{L_{d10} \cdot 12 = 93}$ in

Therefore, there is not sufficient development length for straight bars. Check the hook development length. The base hook development length for 90° hooked No.11 and #10 bars per **LRFD [5.10.8.2.4]** is:

$$L_{hb11} := \frac{38.0 \cdot d_{bar11}}{\sqrt{f'_c}} \qquad\qquad \boxed{L_{hb11} = 28.64} \qquad \text{in}$$

$$L_{hb10} := \frac{38.0 \cdot d_{bar10}}{\sqrt{f'_c}}$$ $\boxed{L_{hb10} = 25.8}$ in

The length available is greater than the base hook development length, therefore the reduction factors do not need to be considered. Hook both the top 9 bars and the bottom layer 5 bars. The remaining 4 bottom layer bars can be terminated 7.75 feet from the inside edge of the bearings at girders 2 and 4, which will allow all bars to be fully developed at this inside edge.

In addition, the tension ties must be spread out sufficiently in the effective anchorage area so that the compressive force on the back face of a CCT Node produced by the development of the ties through bond stress, does not exceed the factored resistance **LRFD [5.8.2.5]**.

Following the steps in E13-1.8.5, we can calculate the nominal resistance based on the limiting compressive stress, (f_{cu}), and the effective cross-section of the back face (Acn_E) at Node E. Analyze for crack control reinforcement being present.

The centroid of the tension ties is $\boxed{centroid_{top} = 5.5}$ inches below the top of the pier cap.

Therefore, the thickness (t_{DE}), and the width (w_{DE}) at the back face are:

$t_{DE} := 2 \cdot centroid_{top}$ $\boxed{t_{DE} = 11.0}$ in

$w_{DE} := W_{cap} \cdot 12$ $\boxed{w_{DE} = 48}$ in

$Acn_E := t_{DE} \cdot w_{DE}$ $\boxed{Acn_E = 528}$ in²

$\boxed{Pu_{DE\_1} = 800.79}$ kips   1-lane loaded controls (Fig. E13-1.8-3)

The capacity at the back face of Node E shall satisfy:

$Pu_{DE\_1} \leq \phi \cdot (0.70 \cdot f'_c) \cdot Acn_E$ ---   (from Sect. 13.7.5)

$\phi = 0.9$ **LRFD [5.5.4.2]** $f'_c = 3.5$ ksi

The factored resistance is:

$Pr_{DE} := \phi \cdot (0.70 \cdot f'_c) \cdot Acn_E$ $\boxed{Pr_{DE} = 1164.24}$ kips

Is $Pr_{DE} \geq Pu_{DE\_1}$? $\boxed{check = \text{"OK"}}$

Because the compressive force on the backface is produced by development of reinforcement, the check as shown above is not necessary **LRFD [5.8.2.5.3b]**.
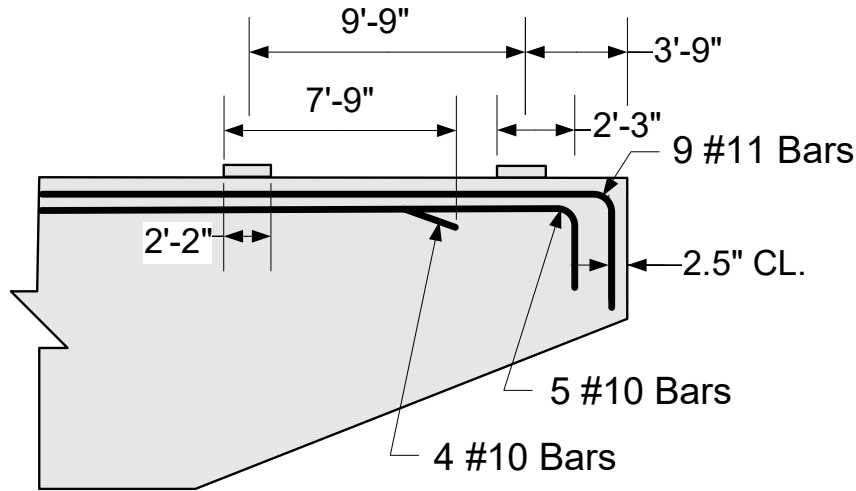
Figure E13-1.8-8
Anchorage of Tension Tie

E13-1.8.8 Provide Crack Control Reinforcement

In the pier cap, the minimum area of crack control reinforcement (As_{crack}) is equal to 0.003 times the width of the member (W_{cap}), and the spacing of the reinforcement (s_v, s_h) in each direction. The spacing of the bars in these grids must not exceed the smaller of d/4 or 12 inches, **LRFD [5.8.2.6]**.

$W_{cap} = 4.0$ ft

d/4 > 12" , therefore s_v and s_h = 12"

$As_{crack} := 0.003 \cdot (12) \cdot W_{cap} \cdot 12$

$\boxed{As_{crack} = 1.73}$ in²

For horizontal reinforcement:

Use 4 - No. 7 horizontal bars at 12 inch spacing in the vertical direction - (Option 1)

$As_{No7} := 0.6013$

$\boxed{4 \cdot As_{No7} = 2.41}$ in²
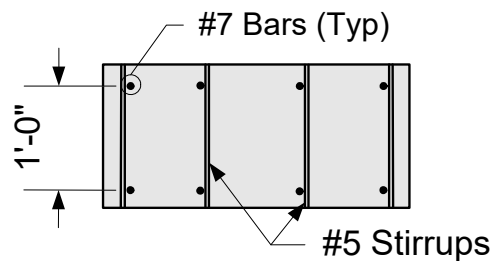
Figure E13-1.8-9
Crack Control Reinforcement - Option 1

OR: If we assume 6-inch vertical spacing - (Option 2)

$As_{crack} := 0.003 \cdot (6) \cdot W_{cap} \cdot 12$

$\boxed{As_{crack} = 0.86}$ in^2

Using 2 - No. 7 horiz. bars at 6 inch spacing

$\boxed{2 \cdot As_{No7} = 1.2}$ in^2

Is $2 \cdot As_{No7} \geq As_{crack}$?

$\boxed{check = \text{"OK"}}$

Therefore, No. 7 bars at 6" vertical spacing, placed horizontally on each side of the cap will satisfy this criteria.
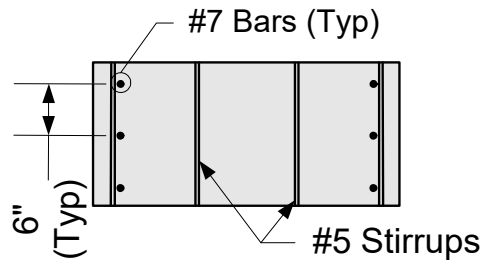
Figure E13-1.8-10
Crack Control Reinforcement - Option 2

This 6-inch spacing for the (No. 7 bars), is also used along the bottom of the cap for temperature and shrinkage reinforcement.

For vertical reinforcement:

The stirrups are spaced at, $\boxed{s_{stir} = 8}$ inches. Therefore the required crack control reinforcement within this spacing is:

$As_{crack2} := 0.003 \cdot (s_{stir}) \cdot W_{cap} \cdot 12$

$\boxed{As_{crack2} = 1.15}$ in^2

4 legs of No.5 stirrups at $\boxed{s_{stir} = 8}$ inch spacing in the horizontal direction

$\boxed{4 \cdot As_{No5} = 1.23}$ in^2

Is $4 \cdot As_{No5} \geq As_{crack2}$?

$\boxed{check = \text{"OK"}}$

Therefore, pairs of (No. 5 bar) double-legged stirrups at 8" horizontal spacing will satisfy this criteria.

E13-1.8.9 Summary of Cap Reinforcement
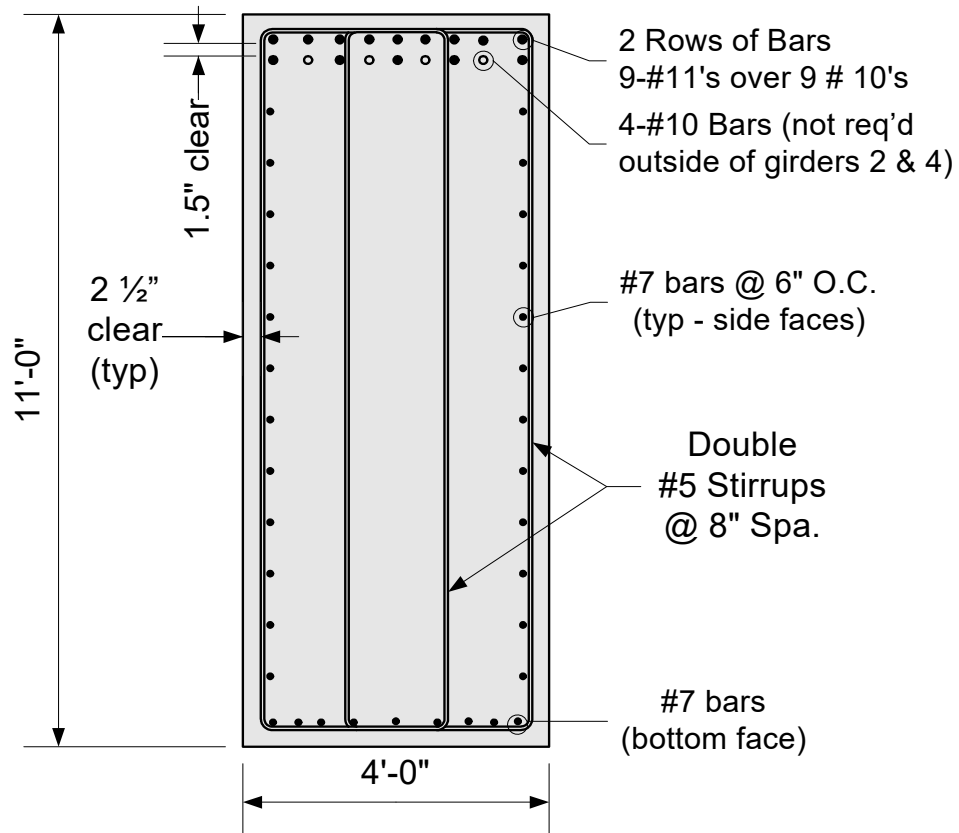
Figure E13-1.8-11
Pier Cap Design Summary

E13-1.9 Design Pier Column

As stated in E13-1.7, the critical section in the pier column is where the column meets the footing, or at the column base. The governing force effects and their corresponding limit states were determined to be:

Strength V

$Ax_{colStrV}$ = 2099.51　kips

$MuT_{colStrV}$ = 8315.32　kip-ft

$MuL_{colStrV}$ = 2369.38　kip-ft

Strength III

$$\boxed{VuT_{col} = 49.86}$$ kips

Strength V

$$\boxed{VuL_{col} = 109.25}$$ kips

A preliminary estimate of the required section size and reinforcement is shown in Figure E13-1.9-1.
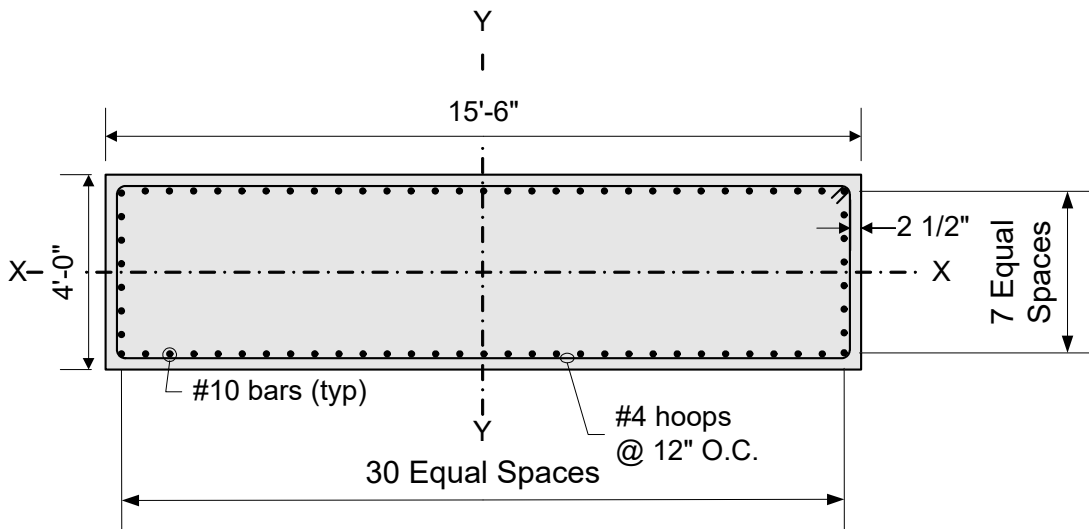
Figure E13-1.9-1
Preliminary Pier Column Design

E13-1.9.1 Design for Axial Load and Biaxial Bending (Strength V):

The preliminary column reinforcing is show in Figure E13-1.9-1 and corresponds to #10 bars equally spaced around the column perimeter. **LRFD [5.6.4.2]** prescribes limits (both maximum and minimum) on the amount of reinforcing steel in a column. These checks are performed on the preliminary column as follows:

$Num\_bars := 74$ $bar\_area10 := 1.27$ in^2 $bar\_dia10 := 1.27$ in

$A_{s\_col} := (Num\_bars) \cdot (bar\_area10)$ $\boxed{A_{s\_col} = 93.98}$ in^2

$A_{g\_col} := (W_{col}) \cdot (L_{col}) \cdot 12^2$ $\boxed{A_{g\_col} = 8928}$ in^2

$\dfrac{A_{s\_col}}{A_{g\_col}} = 0.0105$ $0.0105 \leq 0.08$ (max. reinf. check)   OK

$$\frac{0.135 \cdot f'_c}{f_y} = 0.008 \quad \text{(but need not be greater than 0.015)} \qquad 0.0105 \geq 0.008 \quad \text{(min. reinf. check)} \qquad \text{OK}$$

The column slenderness ratio (Kl_u/r) about each axis of the column is computed below in order to assess slenderness effects. Note that the Specifications only permit the following approximate evaluation of slenderness effects when the slenderness ratio is below 100.

For this pier, the unbraced lengths (l_{ux}, l_{uy}) used in computing the slenderness ratio about each axis is the full pier height. This is the height from the top of the footing to the top of the pier cap (26 feet). The effective length factor in the longitudinal direction, K_x, is taken equal to 2.1. This assumes that the superstructure has no effect on restraining the pier from buckling. In essence, the pier is considered a free-standing cantilever in the longitudinal direction. The effective length factor in the transverse direction, K_y, is taken to equal 1.0.

The radius of gyration (r) about each axis can then be computed as follows:

$$I_{xx} := \frac{(L_{col} \cdot 12) \cdot (W_{col} \cdot 12)^3}{12} \qquad\qquad \boxed{I_{xx} = 1714176} \qquad \text{in}^4$$

$$I_{yy} := \frac{(W_{col} \cdot 12) \cdot (L_{col} \cdot 12)^3}{12} \qquad\qquad \boxed{I_{yy} = 25739424} \qquad \text{in}^4$$

$$r_{xx} := \sqrt{\frac{I_{xx}}{A_{g\_col}}} \qquad\qquad \boxed{r_{xx} = 13.86} \qquad \text{in}$$

$$r_{yy} := \sqrt{\frac{I_{yy}}{A_{g\_col}}} \qquad\qquad \boxed{r_{yy} = 53.69} \qquad \text{in}$$

The slenderness ratio for each axis now follows:

$$K_x := 2.1$$

$$K_y := 1.0$$

$$L_u := (H_{col} + H_{cap}) \cdot 12 \qquad\qquad \boxed{L_u = 312} \qquad \text{in}$$

$$\boxed{\frac{K_x \cdot L_u}{r_{xx}} = 47.28} \qquad\qquad 47.28 < 100 \quad \text{OK}$$

$$\boxed{\frac{K_y \cdot L_u}{r_{yy}} = 5.81} \qquad\qquad 5.81 < 100 \quad \text{OK}$$

LRFD [5.6.4.3] permits the slenderness effects to be ignored when the slenderness ratio is less than 22 for members not braced against side sway. It is assumed in this example that the pier is not braced against side sway in either its longitudinal or transverse directions. Therefore, slenderness will be considered for the pier longitudinal direction only (i.e., about the

"X-X" axis).

In computing the amplification factor that is applied to the longitudinal moment, which is the end result of the slenderness effect, the column stiffness (EI) about the "X-X" axis must be defined. In doing so, the ratio of the maximum factored moment due to permanent load to the maximum factored moment due to total load must be identified (β_d).

From Design Step E13-1.7, it can be seen that the force effects contributing to the longitudinal moment are the live load braking force, the temperature force and wind on the structure and live load. None of these are permanent or long-term loads. Therefore, β_d is taken equal to zero for this design.

$$\beta_d := 0$$

$$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c} \qquad \textbf{LRFD [C5.4.2.7]} \qquad \boxed{E_c = 3587} \qquad \text{ksi}$$

$$\boxed{E_s = 29000.00} \qquad \text{ksi}$$

$$\boxed{I_{xx} = 1714176} \qquad \text{in}^4$$

I_s = Moment of Inertia of longitudinal steel about the centroidal axis (in^4)

$$I_s := \frac{\pi \cdot bar\_dia10^4}{64} \cdot (Num\_bars) + 2 \cdot 31 \cdot (bar\_area10) \cdot 20.37^2 \dots$$
$$+ 4 \cdot (bar\_area10) \cdot 14.55^2 + 4 \cdot (bar\_area10) \cdot 8.73^2 + 4 \cdot (bar\_area10) \cdot 2.91^2$$

$$\boxed{I_s = 34187} \qquad \text{in}^4$$

The column stiffness is taken as the greater of the following two calculations:

$$EI_1 := \frac{\dfrac{E_c \cdot I_{xx}}{5} + E_s \cdot I_s}{1 + \beta_d} \qquad \boxed{EI_1 = 2.22 \times 10^9} \qquad \text{k-in}^2$$

$$EI_2 := \frac{\dfrac{E_c \cdot I_{xx}}{2.5}}{1 + \beta_d} \qquad \boxed{EI_2 = 2.46 \times 10^9} \qquad \text{k-in}^2$$

$$EI := \max(EI_1, EI_2) \qquad \boxed{EI = 2.46 \times 10^9} \qquad \text{k-in}^2$$

The final parameter necessary for the calculation of the amplification factor is the phi-factor for compression. This value is defined as follows:

$$\phi_{axial} := 0.75$$

It is worth noting at this point that when axial load is present in addition to flexure, **LRFD [5.5.4.2]** permits the value of phi to be increased linearly to the value for flexure (0.90) as the section changes from compression controlled to tension controlled as defined in **LRFD [5.6.2.1]**. However, certain equations in the Specification still require the use of the phi factor for axial compression (0.75) even when the increase just described is permitted. Therefore, for

the sake of clarity in this example, if phi may be increased it will be labeled separately from ϕ_{axial} identified above.

$As_{col} := 2.53$ in² per foot, based on #10 bars at 6-inch spacing

$b := 12$ inches $\alpha_1 := 0.85$ (for $f'_c < 10.0\,ksi$)   **LRFD [5.6.2.2]**

$a := \dfrac{As_{col} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b}$ $\boxed{a = 4.25}$ inches

$\beta_1 := 0.85$

$c := \dfrac{a}{\beta_1}$ $\boxed{c = 5.00}$ inches

$d_t := W_{col} \cdot 12 - Cover_{co} - 0.5 - \dfrac{bar\_dia10}{2}$ $\boxed{d_t = 44.37}$ inches

$\varepsilon_c := 0.002$ Upper strain limit for compression controlled sections, $f_y = 60$ ksi   **LRFD [Table C5.6.2.1-1]**

$\varepsilon_t := 0.005$ Lower strain limit for tension controlled sections, for $f_y = 60$ ksi
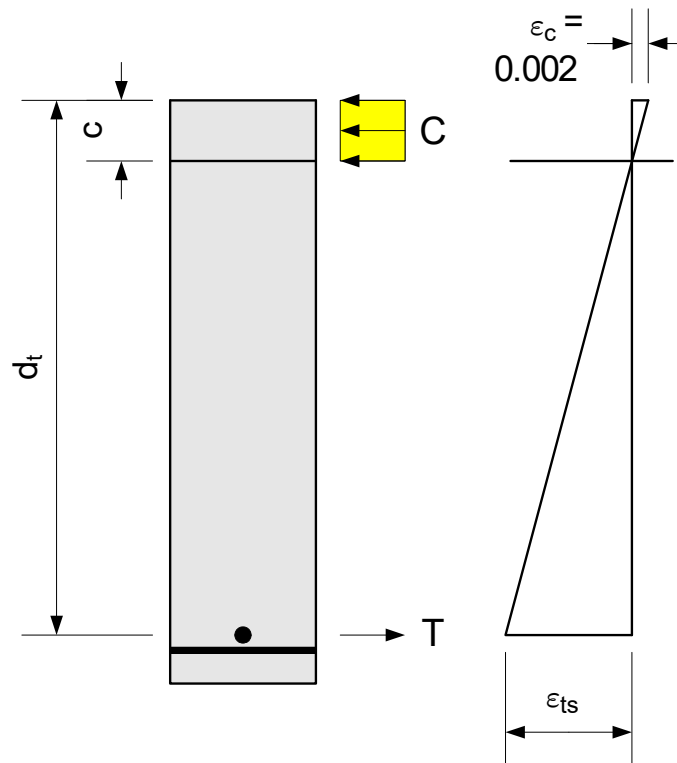
Figure E13-1.9-2
Strain Limit Tension Control Check

$\varepsilon_{ts} := \dfrac{\varepsilon_c}{c} \cdot (d_t - c)$ $\boxed{\varepsilon_{ts} = 0.016}$ $> \varepsilon_t = 0.005$

Therefore, the section is tension controlled and phi shall be equal to 0.9.

$\phi_t := 0.9$

The longitudinal moment magnification factor will now be calculated as follows:

$$P_e := \frac{\pi^2 \cdot EI}{(K_x \cdot L_u)^2}$$

$\boxed{P_e = 56539.53}$ kips

$$\delta_s := \frac{1}{1 - \left(\dfrac{Ax_{colStrV}}{\phi_t \cdot P_e}\right)}$$

$\boxed{\delta_s = 1.04}$

The final design forces at the base of the column for the <u>Strength V</u> limit state will be redefined as follows:

$P_{u\_col} := Ax_{colStrV}$

$\boxed{P_{u\_col} = 2099.51}$ kips

$M_{ux} := MuL_{colStrV} \cdot \delta_s$

$\boxed{M_{ux} = 2471.35}$ kip-ft

$M_{uy} := MuT_{colStrV}$

$\boxed{M_{uy} = 8315.32}$ kip-ft

The assessment of the resistance of a compression member with biaxial flexure for strength limit states is dependent upon the magnitude of the factored axial load. This value determines which of two equations provided by the Specification are used.

If the factored axial load is less than ten percent of the gross concrete strength multiplied by the phi-factor for compression members (ϕ_{axial}), then the Specifications require that a linear interaction equation for only the moments is satisfied (**LRFD [Equation 5.6.4.5-3]**). Otherwise, an axial load resistance (P_{rxy}) is computed based on the reciprocal load method (**LRFD [Equation 5.6.4.5-1]**). In this method, axial resistances of the column are computed (using f_{Low_axial} if applicable) with each moment acting separately (i.e., P_{rx} with M_{ux}, P_{ry} with M_{uy}). These are used along with the theoretical maximum possible axial resistance (P_o multiplied by ϕ_{axial}) to obtain the factored axial resistance of the biaxially loaded column.

Regardless of which of the two equations mentioned in the above paragraph controls, commercially available software is generally used to obtain the moment and axial load resistances.

For this pier design, the procedure as discussed above is carried out as follows:

$\boxed{0.10 \cdot \phi_{axial} \cdot f'_c \cdot A_{g\_col} = 2343.6}$ kips

$\boxed{P_{u\_col} = 2099.51}$ kips $P_{u\_col} < 2343.6K$

Therefore, **LRFD [Equation 5.6.4.5-3]** will be used.

$M_{ux} = 2471.35$ kip-ft $M_{uy} = 8315.32$ kip-ft

The resultant moment equals:

$$Mu := \sqrt{M_{ux}^2 + M_{uy}^2}$$ $Mu = 8674.8$ kip-ft

$M_r := 24052.3$ kip-ft

$$\frac{Mu}{M_r} = 0.36$$ $0.36 \leq 1.0$ OK

The factored flexural resistances shown above, M_r, was obtained by the use of commercial software. This value is the resultant flexural capacity assuming that no axial load is present. Consistent with this, the phi-factor for flexure (0.90) was used in obtaining the factored resistance from the factored nominal strength.

Although the column has a fairly large excess flexural capacity, a more optimal design will not be pursued per the discussion following the column shear check.

E13-1.9.2 Design for Shear (Strength III and Strength V)

The maximum factored transverse and longitudinal shear forces were derived in E13-1.7 and are as follows:

$VuT_{col} = 49.86$ kips   (Strength III)

$VuL_{col} = 109.25$ kips   (Strength V)

These maximum shear forces do not act concurrently. Although a factored longitudinal shear force is present in Strength III and a factored transverse shear force is present in Strength V, they both are small relative to their concurrent factored shear. Therefore, separate shear designs can be carried out for the longitudinal and transverse directions using only the maximum shear force in that direction.

For the pier column of this example, the maximum factored shear in either direction is less than one-half of the factored resistance of the concrete. Therefore, shear reinforcement is not required. This is demonstrated for the longitudinal direction as follows:

$b_v := L_{col} \cdot 12$ $b_v = 186$ in

$h := W_{col} \cdot 12$ $h = 48$ in

Conservatively, d_v may be calculated as shown below, **LRFD [5.7.2.8]**.

$d_v := (0.72) \cdot (h)$ $d_v = 34.56$ in

The above calculation for d_v is simple to use for columns and generally results in a conservative estimate of the shear capacity.

$\beta := 2.0$ $\theta := 45deg$ $\lambda := 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

The nominal concrete shear strength is:

$V_c := 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ **LRFD [5.7.3.3]** $\boxed{V_c = 760.04}$ kips

The nominal shear strength of the column is the lesser of the following two values:

$V_{n1} := V_c$ $\boxed{V_{n1} = 760.04}$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v$ $\boxed{V_{n2} = 5624.64}$ kips

$V_n := \min(V_{n1}, V_{n2})$ $\boxed{V_n = 760.04}$ kips

The factored shear resistance is:

$\phi_v := 0.90$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 684.04}$ kips

$\boxed{\dfrac{V_r}{2} = 342.02}$ kips

$\boxed{VuL_{col} = 109.25}$ kips $\dfrac{V_r}{2} > VuL_{col}$

$\boxed{check = "OK"}$

It has just been demonstrated that transverse steel is not required to resist the applied factored shear forces. However, transverse confinement steel in the form of hoops, ties or spirals is required for compression members. In general, the transverse steel requirements for shear and confinement must both be satisfied per the Specifications.

It is worth noting that although the preceding design checks for shear and flexure show the column to be over designed, a more optimal column size will not be pursued. The reason for this is twofold: First, in this design example, the requirements of the pier cap dictate the column dimensions (a reduction in the column width will increase the moment in the pier cap). Secondly, a short, squat column such as the column in this design example generally has a relatively large excess capacity even when only minimally reinforced.

E13-1.9.3 Transfer of Force at Base of Column

The provisions for the transfer of forces and moments from the column to the footing are new to the AASHTO LRFD Specifications. In general, standard engineering practice for bridge

piers automatically satisfies most, if not all, of these requirements.

In this design example, and consistent with standard engineering practice, all steel reinforcing bars in the column extend into, and are developed, in the footing (see Figure E13-1.12-1). This automatically satisfies the following requirements for reinforcement across the interface of the column and footing: A minimum reinforcement area of 0.5 percent of the gross area of the supported member, a minimum of four bars, and any tensile force must be resisted by the reinforcement. Additionally, with all of the column reinforcement extended into the footing, along with the fact that the column and footing have the same compressive strength, a bearing check at the base of the column and the top of the footing is not applicable.

In addition to the above, the Specifications require that the transfer of lateral forces from the pier to the footing be in accordance with the shear-transfer provisions of **LRFD [5.7.4]**. With the standard detailing practices for bridge piers previously mentioned (i.e., all column reinforcement extended and developed in the footing), along with identical design compressive strengths for the column and footing, this requirement is generally satisfied. However, for the sake of completeness, this check will be carried out as follows:

$A_{cv} := A_{g\_col}$ Area of concrete engaged in shear transfer. $\boxed{A_{cv} = 8928}$ in²

$A_{vf} := A_{s\_col}$ Area of shear reinforcement crossing the shear plane. $\boxed{A_{vf} = 93.98}$ in²

For concrete placed against a clean concrete surface, not intentionally roughened, the following values are obtained from **LRFD [5.7.4.4]**.

$c_{cv} := 0.075$ Cohesion factor, ksi

$\mu := 0.60$ Friction factor

$K_1 := 0.2$

$K_2 := 0.8$

The nominal shear-friction capacity is the smallest of the following three equations (conservatively ignore permanent axial compression):

$V_{nsf1} := c_{cv} \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y$ $\boxed{V_{nsf1} = 4052.88}$ kips

$V_{nsf2} := K_1 \cdot f'_c \cdot A_{cv}$ $\boxed{V_{nsf2} = 6249.6}$ kips

$V_{nsf3} := K_2 \cdot A_{cv}$ $\boxed{V_{nsf3} = 7142.4}$ kips

Define the nominal shear-friction capacity as follows:

$V_{nsf} := min(V_{nsf1}, V_{nsf2}, V_{nsf3})$ $\boxed{V_{nsf} = 4052.88}$ kips

The maximum applied shear was previously identified from the Strength V limit state:

$\boxed{VuL_{col} = 109.25}$ kips

It then follows:

$$\phi_v = 0.9$$

$$\phi_v \cdot (V_{nsf}) = 3647.59 \quad \text{kips}$$

$$\phi_v \cdot (V_{nsf}) \geq VuL_{col}$$

$$\text{check} = \text{"OK"}$$

As can be seen, a large excess capacity exists for this check. This is partially due to the fact that the column itself is over designed in general (this was discussed previously). However, the horizontal forces generally encountered with common bridges are typically small relative to the shear-friction capacity of the column (assuming all reinforcing bars are extended into the footing). In addition, the presence of a shear-key, along with the permanent axial compression from the bridge dead load, further increase the shear-friction capacity at the column/footing interface beyond that shown above.

E13-1.10 Design Pier Piles

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The HP12x53 pile layout used for this pier foundation is shown in Figure E13-1.10-1.

Based on the given pile layout, the controlling limit states for the pile design were given in E13-1.7.3.
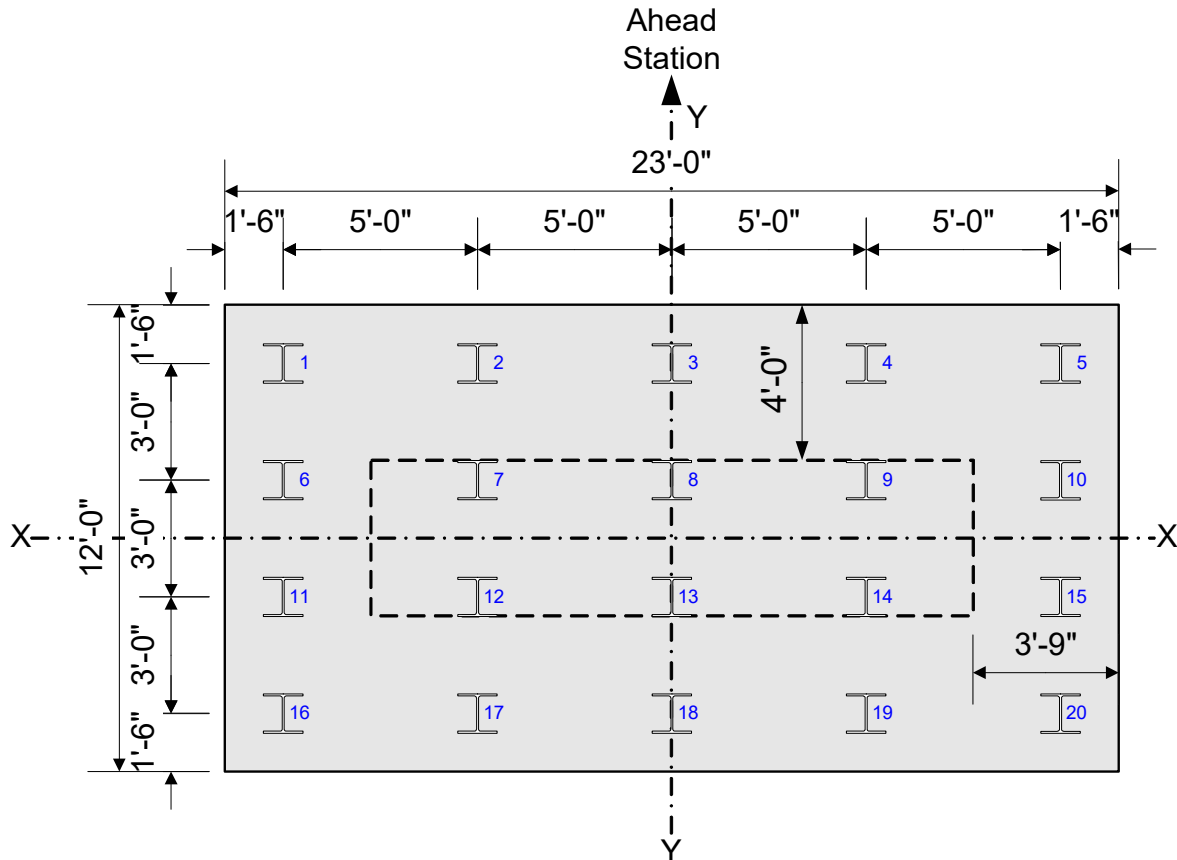
Figure E13-1.10-1
Pier Pile Layout

$N_p := 20$ Number of piles

$$S_{xx} := \frac{10 \cdot 4.5^2 + 10 \cdot 1.5^2}{4.5}$$

$\boxed{S_{xx} = 50}$ ft³

$$S_{yy} := \frac{8 \cdot 10^2 + 8 \cdot 5^2}{10}$$

$\boxed{S_{yy} = 100}$ ft³

Maximum pile reaction (Strength I):

$\boxed{\phi_t = 0.9}$

$\boxed{P_e = 56539.53}$ kips (from column design)

$\boxed{Pu2_{pile\_Str1} = 3179.17}$ kips

$\text{MuT2}_{\text{pile\_Str1}} = 7836.85$ kip-ft

$\text{MuL2}_{\text{pile\_Str1}} = 1856.29$ kip-ft

$$\delta_{\text{pile\_StrI}} := \frac{1}{1 - \left(\dfrac{\text{Pu2}_{\text{pile\_Str1}}}{\phi_t \cdot P_e} \right)}$$

$\delta_{\text{pile\_StrI}} = 1.07$

$$\text{Pu}_p := \frac{\text{Pu2}_{\text{pile\_Str1}}}{N_p} + \frac{\text{MuT2}_{\text{pile\_Str1}}}{S_{yy}} + \frac{\text{MuL2}_{\text{pile\_Str1}} \cdot \delta_{\text{pile\_StrI}}}{S_{xx}}$$

$\text{Pu}_p = 276.93$ kips

$$\text{Pu}_{p\_tons} := \frac{\text{Pu}_p}{2}$$

$\text{Pu}_{p\_tons} = 138.46$ tons

From Wis Bridge Manual, Section 11.3.1.17.6, the vertical pile resistance of HP12x53 pile is :

$\text{Pr}_{12x53} = 110$ tons $\text{check} = \text{"No Good"}$

$\text{Pr}_{12x53\_PDA} = 143$ tons $\text{check} = \text{"OK"}$

Note: PDA with CAPWAP is typically used when it is more economical than modified Gates. This example uses PDA with CAPWAP only to illustrate that vertical pile reactions are satisfied and to minimize example changes due to revised pile values. The original example problem was based on higher pile values than the current values shown in Chapter 11, Table 11.3-5.

Minimum pile reaction (Strength V):

$\text{Pu}_{\text{pile\_StrV}} = 2179.55$ kips

$\text{MuT}_{\text{pile\_StrV}} = 7196.34$ kip-ft

$\text{MuL}_{\text{pile\_StrV}} = 2369.38$ kip-ft

$$\delta_{\text{pile\_StrV}} := \frac{1}{1 - \left(\dfrac{\text{Pu}_{\text{pile\_StrV}}}{\phi_t \cdot P_e} \right)}$$

$\delta_{\text{pile\_StrV}} = 1.04$

$$\text{Pu}_{\text{min\_p}} := \frac{\text{Pu}_{\text{pile\_StrV}}}{N_p} - \frac{\text{MuT}_{\text{pile\_StrV}}}{S_{yy}} - \frac{\text{MuL}_{\text{pile\_StrV}} \cdot \delta_{\text{pile\_StrV}}}{S_{xx}}$$

$$Pu_{min\_p} = -12.49 \quad \text{kips}$$

Capacity for pile uplift is site dependant. Consult with the geotechnical engineer for allowable values.

The horizontal pile resistance of HP12x53 pile from the soils report is :

$$Hr_{12x53} := 14 \quad \text{kips/pile}$$

Pile dimensions in the transverse (xx) and longitudinal (yy) directions:

$$B_{xx} := 12.05 \quad \text{inches}$$

$$B_{yy} := 11.78 \quad \text{inches}$$

Pile spacing in the transverse and longitudinal directions:

$$Spa_{xx} := 5.0 \quad \text{feet}$$

$$\frac{Spa_{xx}}{\dfrac{B_{xx}}{12}} = 4.98 \qquad \text{Say: 5B}$$

$$Spa_{yy} := 3.0 \quad \text{feet}$$

$$\frac{Spa_{yy}}{\dfrac{B_{yy}}{12}} = 3.06 \qquad \text{Say: 3B}$$

Use the pile multipliers from **LRFD [Table 10.7.2.4-1]** to calculate the group resistance of the piles in each direction.

$$Hr_{xx} := Hr_{12x53} \cdot 4 \cdot (1.0 + 0.85 + 0.70 \cdot 3)$$

$$Hr_{xx} = 221.2 \quad \text{kips}$$

$$HuT_{pileStrIII} = 49.86 \quad \text{kips}$$

$$Hr_{xx} \geq HuT_{pileStrIII}$$

$$check = \text{"OK"}$$

$$Hr_{yy} := Hr_{12x53} \cdot 5 \cdot (0.7 + 0.5 + 0.35 \cdot 2)$$

$$Hr_{yy} = 133 \quad \text{kips}$$

$$HuL_{pileStrV} = 109.25 \quad \text{kips}$$

$$Hr_{yy} \geq HuL_{pileStrV}$$

$$check = \text{"OK"}$$

E13-1.11 - Design Pier Footing

In E13-1.7, the <u>Strength I</u> limit states was identified as the governing limit state for the design of the pier footing.

Listed below are the <u>Strength I footing loads</u> for <u>one</u>, <u>two</u> and <u>three lanes loaded</u>:

$\boxed{Pu1_{ftgStr1} = 2643.74}$　　kips　　　　$\boxed{Pu2_{ftgStr1} = 2928.7}$　　kips

$\boxed{MuT1_{ftgStr1} = 7267.81}$　　kip-ft　　　$\boxed{MuT2_{ftgStr1} = 7836.85}$　　kip-ft

$\boxed{MuL1_{ftgStr1} = 1187.7}$　　kip-ft　　　$\boxed{MuL2_{ftgStr1} = 1856.29}$　　kip-ft

$\boxed{Pu3_{ftgStr1} = 3124.66}$　　kips

$\boxed{MuT3_{ftgStr1} = 4541.55}$　　kip-ft

$\boxed{MuL3_{ftgStr1} = 2315.94}$　　kip-ft

The longitudinal moment given above must be magnified to account for slenderness of the column (see E13-1.9). The computed <u>magnification factor</u> and <u>final factored forces</u> are:

$$\delta_{s1\_ftgStr1} := \frac{1}{1 - \left(\dfrac{Pu1_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad\qquad \boxed{\delta_{s1\_ftgStr1} = 1.05}$$

$$\delta_{s2\_ftgStr1} := \frac{1}{1 - \left(\dfrac{Pu2_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad\qquad \boxed{\delta_{s2\_ftgStr1} = 1.06}$$

$$\delta_{s3\_ftgStr1} := \frac{1}{1 - \left(\dfrac{Pu3_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad\qquad \boxed{\delta_{s3\_ftgStr1} = 1.07}$$

$MuL1_{ftgStr1\delta} := \delta_{s1\_ftgStr1} \cdot MuL1_{ftgStr1}$　　　$\boxed{MuL1_{ftgStr1\delta} = 1252.79}$　kip-ft

$MuL2_{ftgStr1\delta} := \delta_{s2\_ftgStr1} \cdot MuL2_{ftgStr1}$　　　$\boxed{MuL2_{ftgStr1\delta} = 1969.65}$　kip-ft

$MuL3_{ftgStr1\delta} := \delta_{s3\_ftgStr1} \cdot MuL3_{ftgStr1}$　　　$\boxed{MuL3_{ftgStr1\delta} = 2467.46}$　kip-ft

The calculations for the Strength I pile loads on the footing are calculated below for one, two and three lanes loaded.

$N_p = 20$ Number of piles

$S_{xx} = 50$ ft³

$S_{yy} = 100$ ft³

The following illustrates the corner pile loads for 2 lanes loaded:

$$Pu2_1 := \frac{Pu2_{ftgStr1}}{N_p} + \frac{MuT2_{ftgStr1}}{S_{yy}} + \frac{MuL2_{ftgStr1\delta}}{S_{xx}}$$ $Pu2_1 = 264.2$ kips

$$Pu2_5 := \frac{Pu2_{ftgStr1}}{N_p} - \frac{MuT2_{ftgStr1}}{S_{yy}} + \frac{MuL2_{ftgStr1\delta}}{S_{xx}}$$ $Pu2_5 = 107.46$ kips

$$Pu2_{16} := \frac{Pu2_{ftgStr1}}{N_p} + \frac{MuT2_{ftgStr1}}{S_{yy}} - \frac{MuL2_{ftgStr1\delta}}{S_{xx}}$$ $Pu2_{16} = 185.41$ kips

$$Pu2_{20} := \frac{Pu2_{ftgStr1}}{N_p} - \frac{MuT2_{ftgStr1}}{S_{yy}} - \frac{MuL2_{ftgStr1\delta}}{S_{xx}}$$ $Pu2_{20} = 28.67$ kips

Pile loads between the corners can be interpolated. Similar calculations for the piles for the cases of one, two and three lanes loaded produce the following results:

$$Pu1 = \begin{pmatrix} 229.92 & 193.58 & 157.24 & 120.9 & 84.56 \\ 213.22 & 176.88 & 140.54 & 104.2 & 67.86 \\ 196.51 & 160.17 & 123.84 & 87.5 & 51.16 \\ 179.81 & 143.47 & 107.13 & 70.79 & 34.45 \end{pmatrix}$$

$$Pu2 = \begin{pmatrix} 264.2 & 225.01 & 185.83 & 146.64 & 107.46 \\ 237.93 & 198.75 & 159.57 & 120.38 & 81.2 \\ 211.67 & 172.49 & 133.3 & 94.12 & 54.94 \\ 185.41 & 146.23 & 107.04 & 67.86 & 28.67 \end{pmatrix}$$

$$Pu3 = \begin{pmatrix} 251 & 228.29 & 205.58 & 182.87 & 160.17 \\ 218.1 & 195.39 & 172.68 & 149.97 & 127.27 \\ 185.2 & 162.49 & 139.78 & 117.08 & 94.37 \\ 152.3 & 129.59 & 106.88 & 84.18 & 61.47 \end{pmatrix}$$

$Pu1_{pile} = 229.92$ kips $Pu2_{pile} = 264.2$ kips $Pu3_{pile} = 251$ kips

A conservative simplification is to use the <u>maximum pile reaction</u> for all piles when calculating the <u>total moment</u> and <u>one way shear forces</u> on the footing.

$Pu := \max(Pu1_{pile}, Pu2_{pile}, Pu3_{pile})$ $Pu = 264.2$ kips

E13-1.11.1 Design for Moment

The footing is designed for moment using the pile forces computed above on a per-foot basis acting on each footing face. The design section for moment is at the face of the column. The following calculations are based on the outer row of piles in each direction, respectively.

$L_{ftg\_xx} := L_{ftg}$ $L_{ftg\_xx} = 23$ feet

$L_{ftg\_yy} := W_{ftg}$ $L_{ftg\_yy} = 12$ feet

Applied factored load per foot in the "X" direction:

$Pu_{Mom\_xx} := Pu \cdot 5$ $Pu_{Mom\_xx} = 1320.98$ kips

$$R_{xx} := \frac{Pu_{Mom\_xx}}{L_{ftg\_xx}}$$

$\boxed{R_{xx} = 57.43}$　　　kips per foot

Estimation of applied factored load per foot in the "Y" direction:

$$Pu_{Mom\_yy} := Pu \cdot 4$$

$\boxed{Pu_{Mom\_yy} = 1056.79}$ kips

$$R_{yy} := \frac{Pu_{Mom\_yy}}{L_{ftg\_yy}}$$

$\boxed{R_{yy} = 88.07}$　　　kips per foot

$\boxed{arm_{xx} := 2.5}$　　feet

$\boxed{arm_{yy} := 2.25}$　　feet

The moment on a per foot basis is then:

$$Mu_{xx} := R_{xx} \cdot arm_{xx}$$

$\boxed{Mu_{xx} = 143.59}$　　kip-ft per foot

$$Mu_{yy} := R_{yy} \cdot arm_{yy}$$

$\boxed{Mu_{yy} = 198.15}$　　kip-ft per foot

Once the maximum moment at the critical section is known, flexure reinforcement must be determined. The footing flexure reinforcement is located in the bottom of the footing and rests on top of the piles.

Assume #8 bars:

$\boxed{bar\_diam8 := 1.0}$　　inches

$\boxed{bar\_area8 := 0.79}$　　in²

$\boxed{f_y = 60}$　　ksi

The footing minimum tensile reinforcement requirements will be calculated. The tensile reinforcement provided must be enough to develop a factored flexural resistance at least equal to the lesser of the cracking strength or 1.33 times the factored moment from the applicable strength load combinations, **LRFD [5.6.3.3]**.

The cracking strength is calculated as follows, **LRFD[5.6.3.3]**:
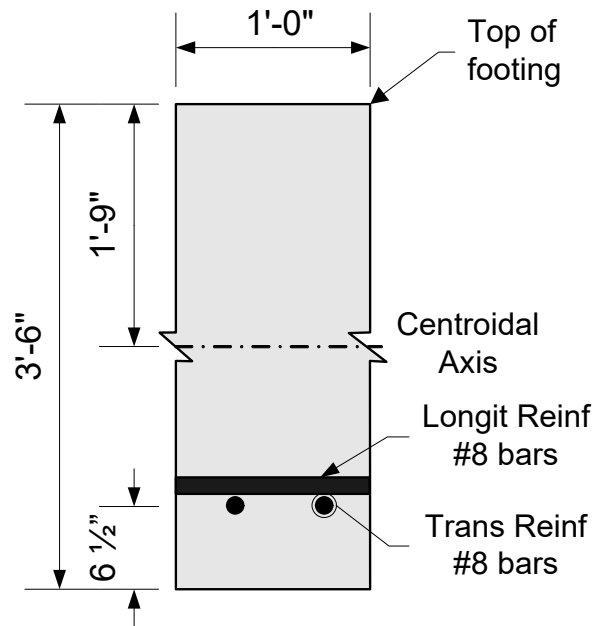
Figure E13-1.11-1
Footing Cracking Moment Dimensions

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.45}$ ksi

$S_g := \dfrac{b \left(H_{ftg} \cdot 12 \right)^2}{6}$ $\boxed{S_g = 3528}$ in⁴

$y_t := \dfrac{H_{ftg} \cdot 12}{2}$ $\boxed{y_t = 21}$ in

$M_{cr} = \gamma_3 \left(\gamma_1 \cdot f_r \right) S_g$ therefore, $M_{cr} = 1.1 (f_r) S_g$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement
<u>for A615, Grade 60 reinforcement</u>

$M_{cr} := 1.1 f_r \cdot S_g \cdot \dfrac{1}{12}$ $\boxed{M_{cr} = 145.21}$ kip-ft

1.33 times the factored controlling footing moment is:

$Mu_{ftg} := max(Mu_{xx}, Mu_{yy})$ $\boxed{Mu_{ftg} = 198.15}$ kip-ft

$\boxed{1.33 \cdot Mu_{ftg} = 263.54}$ kip-ft

$M_{Design} := min(M_{cr}, 1.33 \cdot Mu_{ftg})$ $\boxed{M_{Design} = 145.21}$ kip-ft

Mu_{ftg} exceeds M_{Design}, therefore set $M_{Design} = Mu_{ftg}$

Since the transverse moment controlled, M_{yy}, detail the transverse reinforcing to be located directly on top of the piles.

Effective depth, d_e = total footing thickness - cover - 1/2 bar diameter

$d_e := H_{ftg} \cdot 12 - Cover_{fb} - \dfrac{bar\_diam8}{2}$ $\boxed{d_e = 35.5}$ in

Solve for the required amount of reinforcing steel, as follows:

$\boxed{\phi_f := 0.90}$

$\boxed{b = 12}$ in

$\boxed{f'_c = 3.5}$ ksi

$Rn := \dfrac{M_{Design} \cdot 12}{\phi_f \cdot b \cdot d_e^2}$ $\boxed{Rn = 0.175}$

$\rho := 0.85 \left(\dfrac{f'_c}{f_y}\right) \cdot \left(1.0 - \sqrt{1.0 - \dfrac{2 \cdot Rn}{0.85 \cdot f'_c}}\right)$ $\boxed{\rho = 0.00300}$

$As_{ftg} := \rho \cdot b \cdot d_e$ $\boxed{As_{ftg} = 1.28}$ in² per foot

Required bar spacing = $\boxed{\dfrac{bar\_area8}{As_{ftg}} \cdot 12 = 7.41}$ in

Use #8 bars @ $bar\_space := 7$

$A_{sftg} := bar\_area8 \cdot \left(\dfrac{12}{bar\_space}\right)$ $\boxed{A_{sftg} = 1.35}$ in² per foot

Is $A_{sftg} \geq As_{ftg}$? $\boxed{check = "OK"}$

Similar calculations can be performed for the reinforcing in the longitudinal direction. The effective depth for this reinforcing is calculated based on the longitudinal bars resting directly on top of the transverse bars.

E13-1.11.2 Punching Shear Check

The factored force effects from E13-1.7 for the punching shear check at the column are:

$Pu3_{ftgStr1} = 3124.66$ kips

$MuT3_{ftgStr1} = 4541.55$ kip-ft

$MuL3_{ftgStr1\delta} = 2467.46$ kip-ft

$$Pu3 = \begin{pmatrix} 251 & 228.29 & 205.58 & 182.87 & 160.17 \\ 218.1 & 195.39 & 172.68 & 149.97 & 127.27 \\ 185.2 & 162.49 & 139.78 & 117.08 & 94.37 \\ 152.3 & 129.59 & 106.88 & 84.18 & 61.47 \end{pmatrix}$$

$Pu3_{pile} = 251$ kips

With the applied factored loads determined, the next step in the column punching shear check is to define the critical perimeter, b_o. The Specifications require that this perimeter be minimized, but need not be closer than $d_v/2$ to the perimeter of the concentrated load area. In this case, the concentrated load area is the area of the column on the footing as seen in plan.

The effective shear depth, d_v, must be defined in order to determine b_o and the punching (or two-way) shear resistance. An average effective shear depth should be used since the two-way shear area includes both the "X-X" and "Y-Y" sides of the footing. In other words, d_{ex} is not equal to d_{ey}, therefore d_{vx} will not be equal to d_{vy}. This is illustrated as follows assuming a 3'-6" footing with #8 reinforcing bars at 6" on center in both directions in the bottom of the footing:

	$b = 12$	in
$h_{ftg} := H_{ftg} \cdot 12$	$h_{ftg} = 42$	in
$A_{s\_ftg} := 2 \cdot (bar\_area8)$	$A_{s\_ftg} = 1.58$	in² per foot width

Effective depth for each axis:

$Cover_{fb} = 6$ in

$d_{ey} := h_{ftg} - Cover_{fb} - \dfrac{bar\_diam8}{2}$ $d_{ey} = 35.5$ in

$d_{ex} := h_{ftg} - Cover_{fb} - bar\_diam8 - \dfrac{bar\_diam8}{2}$ $d_{ex} = 34.5$ in

Effective shear depth for each axis:

$$T_{ftg} := A_{s\_ftg} \cdot f_y \qquad\qquad \boxed{T_{ftg} = 94.8} \qquad \text{kips}$$

$$a_{ftg} := \frac{T_{ftg}}{\alpha_1 \cdot f'_c \cdot b} \qquad\qquad \boxed{a_{ftg} = 2.66} \qquad \text{in}$$

$$d_{vx} := \max\left(d_{ex} - \frac{a_{ftg}}{2}, 0.9 \cdot d_{ex}, 0.72 \cdot h_{ftg}\right) \qquad \boxed{d_{vx} = 33.17} \qquad \text{in}$$

$$d_{vy} := \max\left(d_{ey} - \frac{a_{ftg}}{2}, 0.9 \cdot d_{ey}, 0.72 \cdot h_{ftg}\right) \qquad \boxed{d_{vy} = 34.17} \qquad \text{in}$$

Average effective shear depth:

$$d_{v\_avg} := \frac{d_{vx} + d_{vy}}{2} \qquad\qquad \boxed{d_{v\_avg} = 33.67} \quad \text{in}$$

With the average effective shear depth determined, the critical perimeter can be calculated as follows:

$$b_{col} := L_{col} \cdot 12 \qquad\qquad \boxed{b_{col} = 186} \qquad \text{in}$$

$$t_{col} := W_{col} \cdot 12 \qquad\qquad \boxed{t_{col} = 48} \qquad \text{in}$$

$$b_o := 2\left[b_{col} + 2 \cdot \left(\frac{d_{v\_avg}}{2}\right)\right] + 2 \cdot \left[t_{col} + 2 \cdot \left(\frac{d_{v\_avg}}{2}\right)\right] \qquad \boxed{b_o = 602.69} \qquad \text{in}$$

The <u>factored shear resistance</u> to <u>punching shear</u> is the smaller of the following two computed values: **LRFD [5.12.8.6.3]**

$$\beta_c := \frac{b_{col}}{t_{col}} \qquad\qquad \boxed{\beta_c = 3.88}$$

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_{n\_punch1} := \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \lambda\sqrt{f'_c} \cdot (b_o) \cdot (d_{v\_avg}) \qquad \boxed{V_{n\_punch1} = 3626.41} \quad \text{kips}$$

$$V_{n\_punch2} := 0.126 \cdot \left(\lambda\sqrt{f'_c}\right) \cdot (b_o) \cdot (d_{v\_avg}) \qquad \boxed{V_{n\_punch2} = 4783.77} \quad \text{kips}$$

$$V_{n\_punch} := \min\left(V_{n\_punch1}, V_{n\_punch2}\right) \qquad \boxed{V_{n\_punch} = 3626.41} \quad \text{kips}$$

$$\phi_v = 0.9$$

$$V_{r\_punch} := \phi_v \cdot (V_{n\_punch}) \qquad\qquad \boxed{V_{r\_punch} = 3263.77} \quad \text{kips}$$

With the factored shear resistance determined, the applied <u>factored punching shear load</u> will be computed. This value is obtained by summing the loads in the piles that are outside of the critical perimeter. As can be seen in Figure E13-1.11-2, this includes Piles 1 through 5, 6, 10,11, 15, and 16 through 20. These piles are entirely outside of the critical perimeter. If part

of a pile is inside the critical perimeter, then only the portion of the pile load outside the critical perimeter is used for the punching shear check, **LRFD [5.12.8.6.1]**.

$$\left(\frac{t_{col}}{2} + \frac{d_{v\_avg}}{2}\right) \cdot \frac{1}{12} = 3.4 \qquad \text{feet}$$
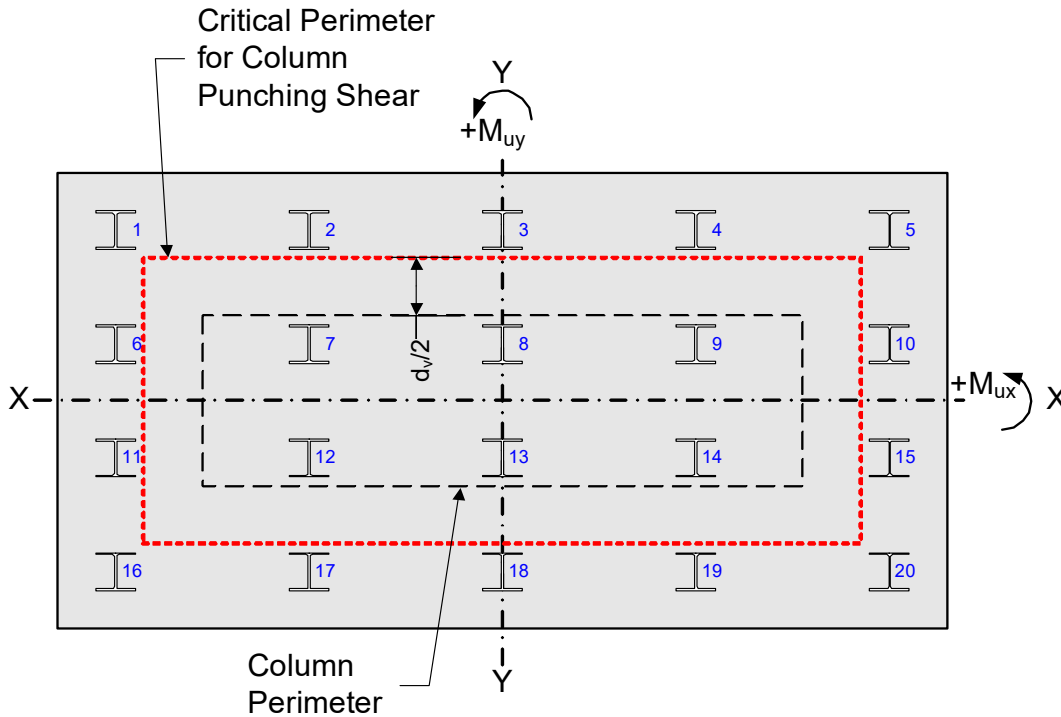
Figure E13-1.11-2
Critical Perimeter for Column Punching Shear

The total applied factored shear used for the punching shear check is the sum of the piles outside of the shear perimeter (1 through 5, 6, 10, 11, 15 and 16 through 20):

$$V_{u\_punch} := max\left(Pu1_{punch\_col}, Pu2_{punch\_col}, Pu3_{punch\_col}\right)$$

$V_{u\_punch} = 2187.26$ kips

$V_{r\_punch} = 3263.77$ kips

$V_{u\_punch} \leq V_{r\_punch}$

check = "OK"

For <u>two-way action</u> around the <u>maximum loaded pile</u>, the pile critical perimeter, b_o, is located a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge.

<u>Two-way action</u> should be checked for the <u>maximum loaded pile</u>, The effective shear depth, d_v, is the same as that used for the punching shear check for the column.

$$V_{u2way} := Pu2_{pile}$$

$V_{u2way} = 264.2$	kips

$d_{v\_avg} = 33.67$	in

$0.5 \cdot d_{v\_avg} = 16.84$	in

Two-way action or punching shear resistance for sections without transverse reinforcement can then be calculated as follows: **LRFD [5.12.8.6.3]**

$\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_n = \left(0.063 + \frac{0.126}{\beta_c} \right) \cdot \lambda \sqrt{f'_c} \cdot b_o \cdot d_v \leq 0.126 \cdot \lambda \sqrt{f'_c} \cdot b_o \cdot d_v$$

$B_{xx} = 12.05$	in

$B_{yy} = 11.78$	in
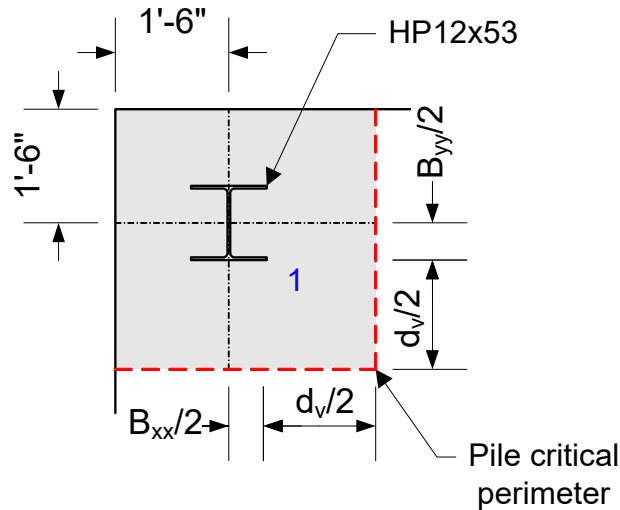
Figure E13-1.11-3
Pile Two-way Action Critical Perimeter

Since the critical section is outside of the footing, only include the portion of the shear perimeter that is located within the footing:

$$b_{o\_xx} := 1.5 \cdot 12 + \frac{B_{xx}}{2} + \frac{d_{v\_avg}}{2}$$

$b_{o\_xx} = 40.86$	in

$$b_{o\_yy} := 1.5 \cdot 12 + \frac{B_{yy}}{2} + \frac{d_{v\_avg}}{2}$$

$b_{o\_yy} = 40.73$	in

Ratio of long to short side of critical perimeter:

$$\beta_{c\_pile} := \frac{b_{o\_xx}}{b_{o\_yy}}$$
$\boxed{\beta_{c\_pile} = 1.003}$

$$b_{o\_pile} := b_{o\_xx} + b_{o\_yy}$$
$\boxed{b_{o\_pile} = 81.59}$ in

$$V_{n\_pile1} := \left(0.063 + \frac{0.126}{\beta_{c\_pile}}\right) \cdot \lambda\sqrt{f'_c} \cdot (b_{o\_pile}) \cdot (d_{v\_avg})$$ $\boxed{V_{n\_pile1} = 969.24}$ kips

$$V_{n\_pile2} := 0.126 \cdot \left(\lambda\sqrt{f'_c}\right) \cdot (b_{o\_pile}) \cdot (d_{v\_avg})$$
$\boxed{V_{n\_pile2} = 647.59}$ kips

$$V_{n\_pile} := \min\left(V_{n\_pile1}, V_{n\_pile2}\right)$$
$\boxed{V_{n\_pile} = 647.59}$ kips

$$\phi_v = 0.9$$

$$V_{r\_pile} := \phi_v \cdot (V_{n\_pile})$$
$\boxed{V_{r\_pile} = 582.83}$ kips

$\boxed{V_{u2way} = 264.2}$ kips

$$V_{r\_pile} \geq V_{u2way}$$
$\boxed{check = \text{"OK"}}$

E13-1.11.3 One Way Shear Check

Design for <u>one way shear</u> in both the <u>transverse</u> and <u>longitudinal directions</u>.

For one way action in the pier footing, in accordance with **LRFD[5.12.8.6.1]** & **[5.7.3.2]** the critical section is taken as the larger of:

    $0.5 \cdot d_v \cdot \cot\theta$ or d_v

    $\theta := 45\deg$

The term d_v is calculated the same as it is for the punching shear above:

$\boxed{d_{vx} = 33.17}$ in

$\boxed{d_{vy} = 34.17}$ in

Now the critical section can be calculated:

$$dv_{xx} := \max\left(0.5 \cdot d_{vx} \cdot \cot(\theta), d_{vx}\right)$$ $\boxed{dv_{xx} = 33.17}$ in

$$dv_{yy} := \max\left(0.5 \cdot d_{vy} \cdot \cot(\theta), d_{vy}\right)$$ $\boxed{dv_{yy} = 34.17}$ in

Distance from face of column to CL of pile in longitudinal and transverse directions:

$$\boxed{\text{arm}_{xx} = 2.5} \quad \text{feet}$$

$$\boxed{\text{arm}_{yy} = 2.25} \quad \text{feet}$$

Distance from face of column to outside edge of pile in longitudinal and transverse directions:

$$\boxed{\text{arm}_{xx} \cdot 12 + \frac{B_{yy}}{2} = 35.89} \quad \text{in} \qquad > d_{vx}, \text{design check required}$$

$$\boxed{\text{arm}_{yy} \cdot 12 + \frac{B_{xx}}{2} = 33.02} \quad \text{in} \qquad < d_{vy}, \text{no design check required}$$
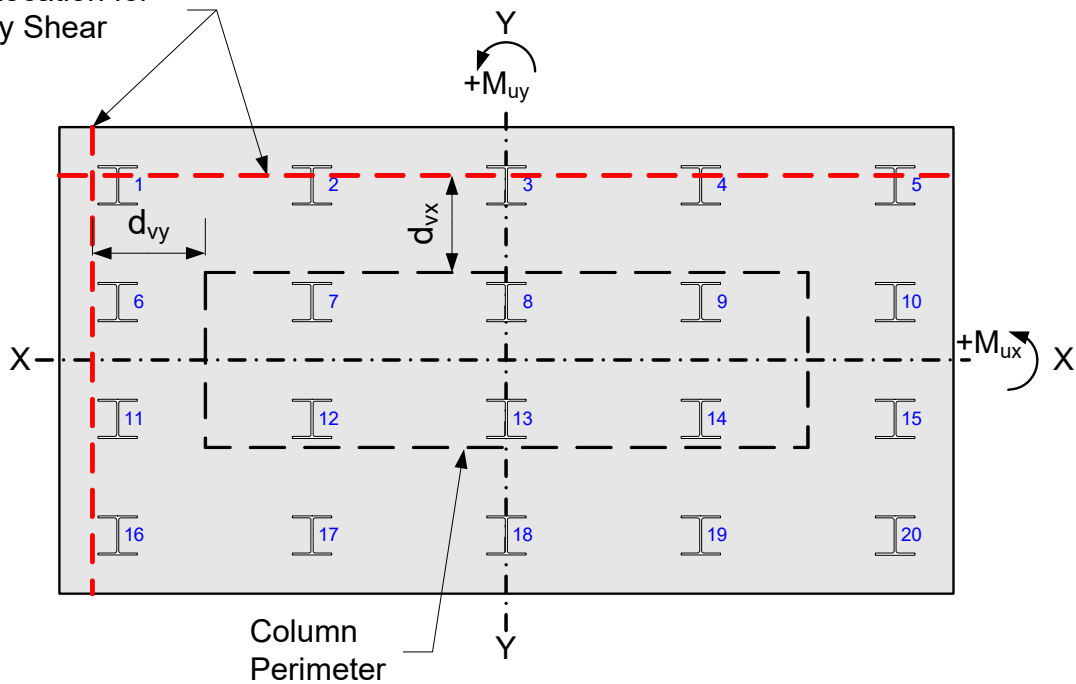
Figure E13-1.11-4
Critical Section for One-Way Shear

Portion of pile outside of the critical section for one way shear in the longitudinal direction:

$$b_{xx} := \text{arm}_{xx} \cdot 12 + \frac{B_{yy}}{2} - d_{vx} \qquad\qquad \boxed{b_{xx} = 2.72} \qquad \text{inches}$$

The load applied to the critical section will be based on the proportion of the pile located outside of the critical section. As a conservative estimate, the maximum pile reaction will be assumed for all piles.

$\boxed{Pu = 264.2}$ kips

$Pu_{1wayx} := Pu \cdot 5$ $\boxed{Pu_{1wayx} = 1320.98}$ kips

$Vu_{1wayx} := Pu_{1wayx} \cdot \dfrac{b_{xx}}{B_{yy}}$ $\boxed{Vu_{1wayx} = 304.76}$ kips

The nominal shear resistance shall be calculated in accordance with **LRFD [5.7.3.3]** and is the lesser of the following:

$\beta_{1way} := 2.0$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$b_v := L_{ftg} \cdot 12$ $\boxed{b_v = 276}$ inches

$V_{n\_1way1} := 0.0316 \cdot \beta_{1way} \cdot \lambda \sqrt{f'_c} \cdot (b_v) \cdot (d_{vx})$ $\boxed{V_{n\_1way1} = 1082.52}$ kips

$V_{n\_1ay2} := 0.25 \cdot (f'_c) \cdot (b_v) \cdot (d_{vx})$ $\boxed{V_{n\_1ay2} = 8011.1}$ kips

$V_{n\_1way} := \min(V_{n\_1way1}, V_{n\_1ay2})$ $\boxed{V_{n\_1way} = 1082.52}$ kips

$\phi_v = 0.9$

$V_{r\_1way} := \phi_v \cdot (V_{n\_1way})$ $\boxed{V_{r\_1way} = 974.27}$ kips

$\boxed{Vu_{1wayx} = 304.76}$ kips

$V_{r\_1way} \geq Vu_{1wayx}$

$\boxed{check = \text{"OK"}}$

E13-1.12 Final Pier Schematic

Figure E13-1.12-1 shows the final pier dimensions along with the required reinforcement in the pier cap and column.
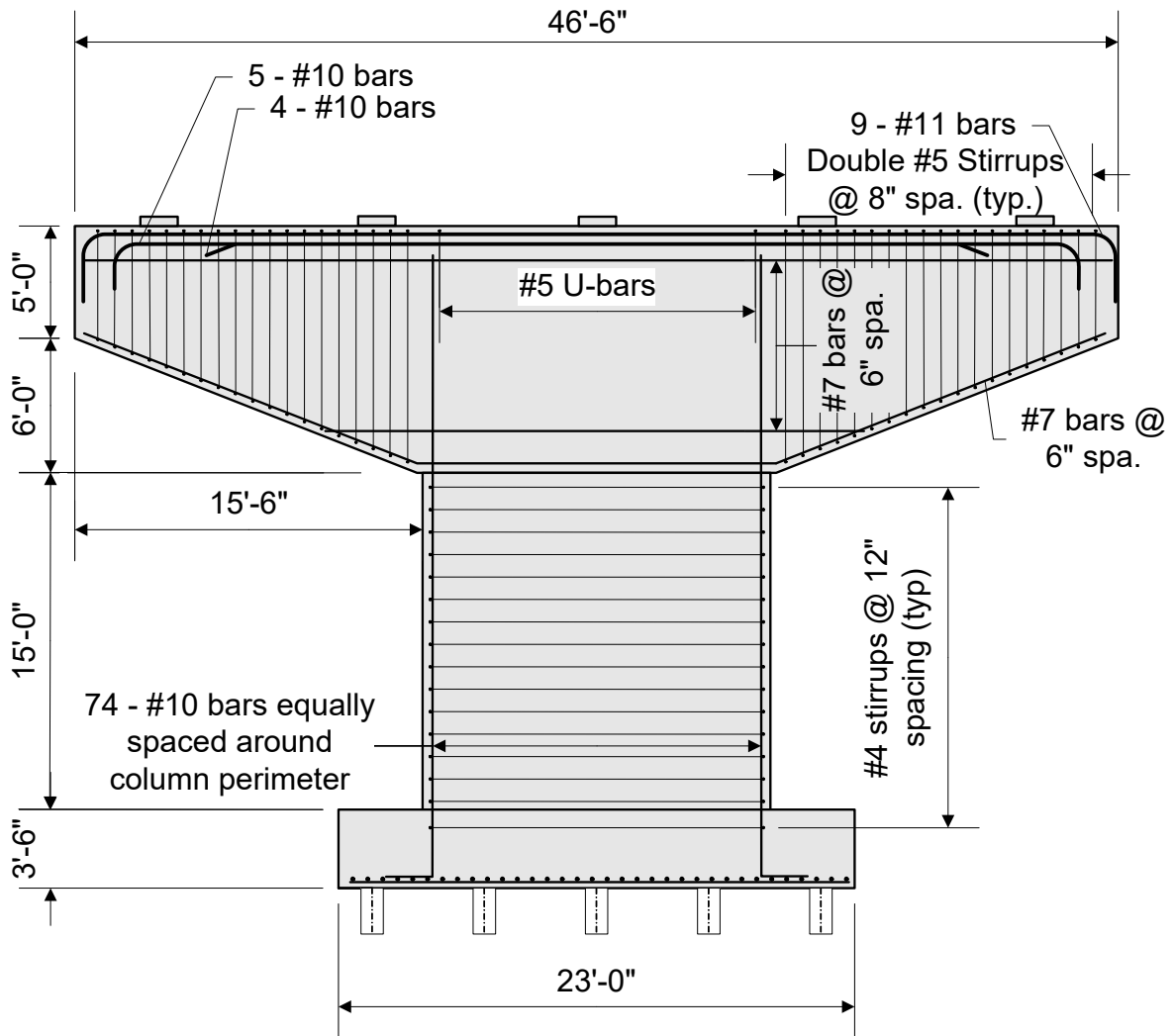
Figure E13-1.12-1
Final Pier Design

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E13-2   Multi-Column Pier Design Example - LRFD

2 Span Bridge, 54W, LRFD Design
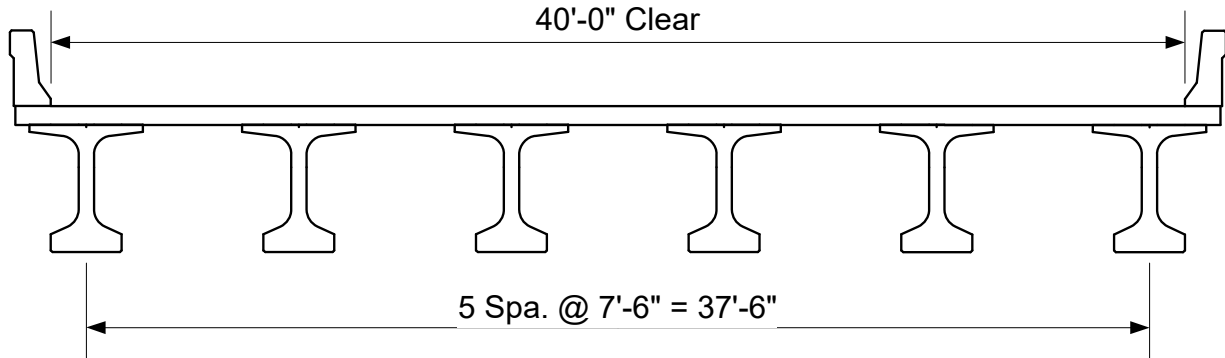
This pier is designed for the superstructure as detailed in example **E19-2**. This is a two-span prestressed girder grade separation structure. Semi-expansion bearings are located at the abutments, and fixed bearings are used at the pier.
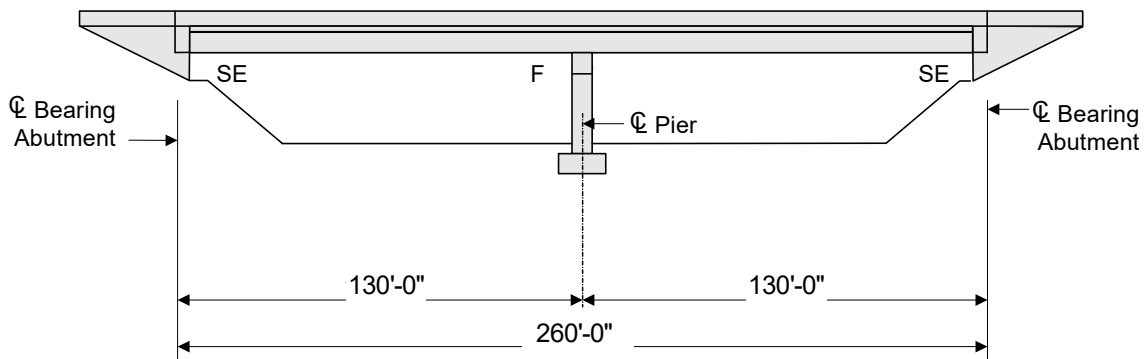
E13-2.1 Obtain Design Criteria

This multi-column pier design example is based on **AASHTO LRFD Bridge Design Specifications, (Ninth Edition - 2020).** The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice. Calculations are only shown for the pier cap. For example column and footing calculations, see example E13-1.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-2.1.1 Material Properties:

$w_c := 0.150$ Concrete density, kcf

$f'_c := 3.5$ Concrete 28-day compressive strength, ksi
LRFD [5.4.2.1 & Table C5.4.2.1-1]

$f_y := 60$ Reinforcement strength, ksi **LRFD [5.4.3 & 6.10.1.7]**

$E_s := 29000$ Modulus of Elasticity of the reinforcing steel, ksi

$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ **LRFD [C5.4.2.4]**

$\boxed{E_c = 3587}$ Modulus of Elasticity of the Concrete, ksi

E13-2.1.2 Reinforcing steel cover requirements (assume epoxy coated bars)

Cover dimension listed below is in accordance with **LRFD [Table 5.10.1-1].**

$Cover_{cap} := 2.5$ Concrete cover in pier cap, inches

E13-2.1.3 Relevant Superstructure Data

$L := 130$ design span length, feet

$w_b := 42.5$ out to out width of deck, feet

$w_{deck} := 40$ clear width of deck, feet

$w_p := 0.387$ weight of Wisconsin Type LF parapet, klf

$t_s := 8$ slab thickness, inches

$t_{haunch} := 4$ haunch thickness, inches

$skew := 0$ skew angle, degrees

$S := 7.5$ girder spacing, ft

$ng := 6$ number of girders

$DOH := \dfrac{w_b - (ng - 1) \cdot S}{2}$ deck overhang length $\boxed{DOH = 2.5}$ feet

$w_{tf} := 48$ width of 54W girder top flange, inches

$t_{tf} := 3$ thickness of 54W girder top flange, inches

$$\text{tf}_{slope} := \frac{2.5}{20.75}$$ slope of bottom surface of top flange $\boxed{\text{tf}_{slope} = 0.12}$ feet per foot

$$\text{girder}_H := 54$$ height of 54W girder, inches

E13-2.1.4 Select Optimum Pier Type

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. The most common pier types are single column (i.e., "hammerhead"), solid wall type, and bent type (multi-column or pile bent). For this design example, a multi-column pier was chosen.

E13-2.1.5 Select Preliminary Pier Dimensions

Since the Specifications do not have standards regarding maximum or minimum dimensions for a pier cap, column, or footing, the designer should base the preliminary pier dimensions on state specific standards, previous designs, and past experience. The pier cap, however, must be wide enough to accommodate the bearing.

$$\text{cap}_L := 41.5$$ overall cap length, ft

$$\text{cap}_H := 4.0$$ pier cap height, ft

$$\text{cap}_W := 3.5$$ pier cap width, ft

$$\text{col}_{spa} := 18.25$$ column spacing, ft

$$\text{col}_d := 3$$ column depth (perpendicular to pier CL), ft

$$\text{col}_w := 4$$ column width (parallel to pier CL), ft

$$\text{col}_h := 18$$ column height, ft

$$\text{cap}_{OH} := 2.5$$ pier cap overhang dimension, ft

Figures E13-2.1-1 and E13-2.1-2 show the preliminary dimensions selected for this pier design example.
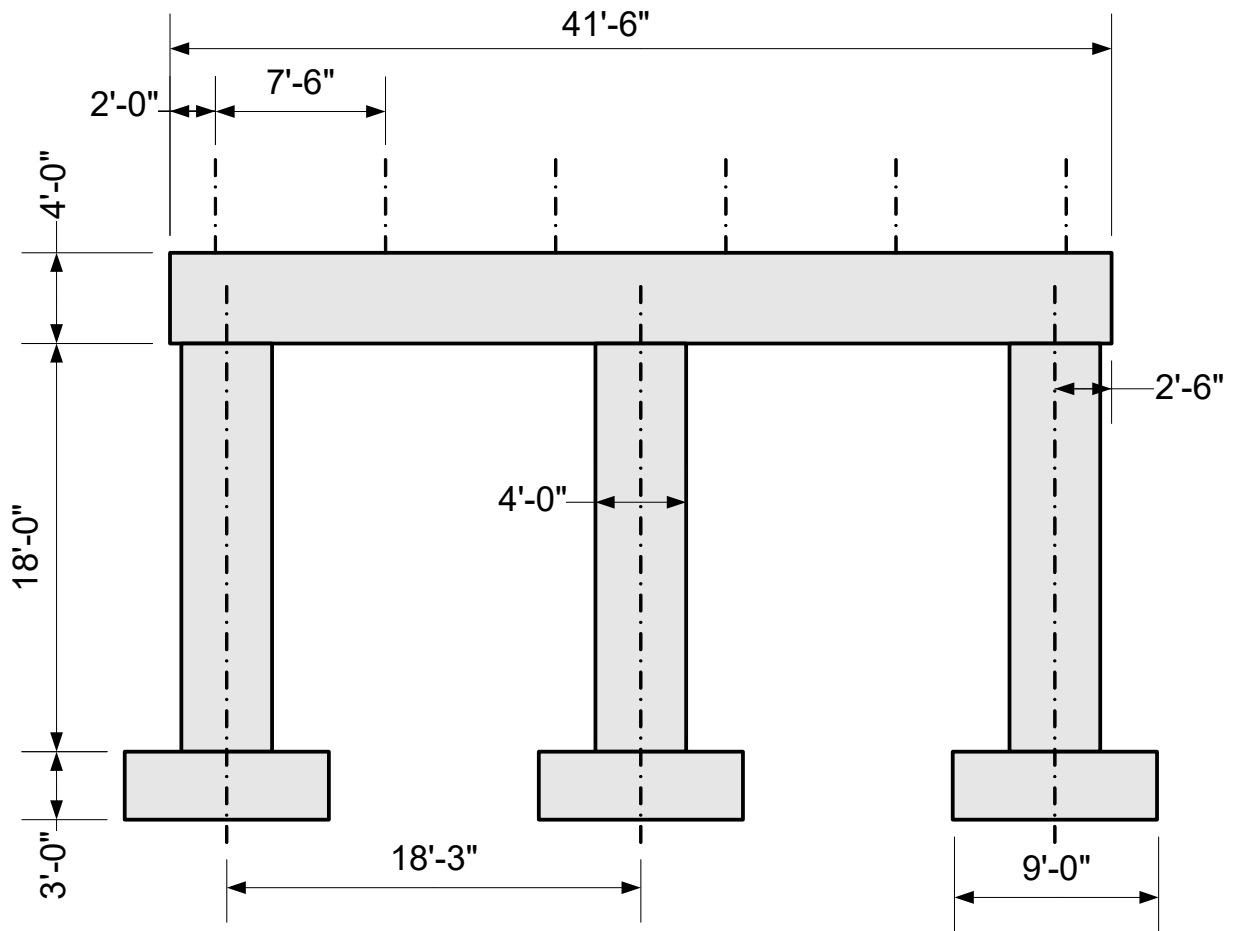
Figure E13-2.1-1
Preliminary Pier Dimensions - Front Elevation
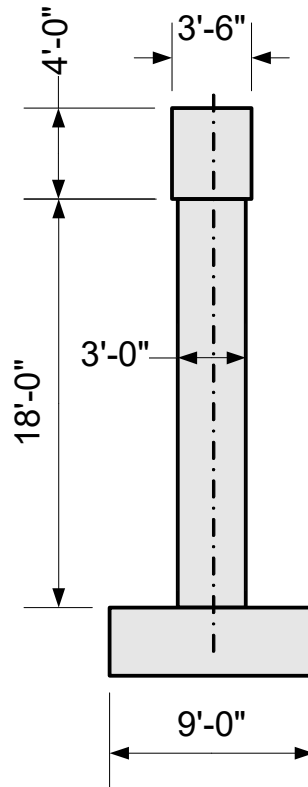
Figure E13-2.1-2
Preliminary Pier Dimensions - End Elevation

E13-2.2 Loads

$w_g := 0.831$ weight of 54W girder, klf

$w_{deck\_int} := w_c \cdot \dfrac{t_s \cdot S}{12}$ weight of deck slab (int), klf $\boxed{w_{deck\_int} = 0.75}$ klf

$OH := DOH - \dfrac{w_{tf}}{2 \cdot 12}$ deck overhang projection, ft $\boxed{OH = 0.5}$ ft

weight of deck slab (ext), klf

$$w_{deck\_ext} := w_c \cdot \left[\frac{t_s}{12} \cdot \left(\frac{S}{2} + DOH \right) + \frac{1}{2} \cdot (OH) \cdot \left(\frac{t_{haunch} + t_{tf}}{12^2} - OH \cdot tf_{slope} \cdot \frac{1}{2} \right) \right]$$

$\boxed{w_{deck\_ext} = 0.63}$ klf

weight of haunch, klf

$w_h := w_c \cdot \dfrac{t_{haunch} \cdot w_{tf}}{12^2}$ $\boxed{w_h = 0.2}$ klf

$w_{diaph\_int} := 0.410$ weight of diaphragms on interior girder (assume 2), kips

$w_{diaph\_ext} := 0.205$ weight of diaphragms on exterior girder, kips

$w_{ws} := 0.020$ future wearing surface, ksf

$w_p := 0.387$ weight of each parapet, klf

weight of concrete diaphragm between exterior girders

$w_{diaph} := w_c \cdot \dfrac{girder_H}{12} \cdot 2$ $\boxed{w_{diaph} = 1.35}$ klf

weight of cap

$w_{cap} := w_c \cdot cap_W \cdot cap_H$ $\boxed{w_{cap} = 2.1}$ klf

E13-2.2.1 Superstructure Dead Loads

DC Loads and Reactions

Interior DC1,DC2 and DW Loads

$w_{DC1\_int} := w_g + w_{deck\_int} + w_h + w_{diaph\_int}$ $\boxed{w_{DC1\_int} = 2.19}$ klf

$$w_{DC2} := \frac{2 \cdot w_p}{ng}$$

$\boxed{w_{DC2} = 0.13}$ klf

$$w_{DW} := \frac{w_{ws} \cdot w_{deck}}{ng}$$

$\boxed{w_{DW} = 0.13}$ klf

Interior DC and DW Reactions

$$R_{DCi} := \left(\frac{1}{2} \cdot L \cdot w_{DC1\_int} + \frac{5}{8} \cdot L \cdot w_{DC2}\right) \cdot 2$$

$\boxed{R_{DCi} = 305.79}$ kips

$$R_{DWi} := \left(\frac{5}{8} \cdot L \cdot w_{DW}\right) \cdot 2$$

$\boxed{R_{DWi} = 21.67}$ kips

Exterior DC1 Loads

$$w_{DC1\_ext} := w_g + w_{deck\_ext} + w_h + w_{diaph\_ext}$$

$\boxed{w_{DC1\_ext} = 1.86}$ klf

Note: DC2 and DW loads are the same for interior and exterior girders.

Exterior DC and DW Reactions

$$R_{DCe} := \left(\frac{1}{2} \cdot L \cdot w_{DC1\_ext} + \frac{5}{8} \cdot L \cdot w_{DC2}\right) \cdot 2$$

$\boxed{R_{DCe} = 262.98}$ kips

$$R_{DWe} := \left(\frac{5}{8} \cdot L \cdot w_{DW}\right) \cdot 2$$

$\boxed{R_{DWe} = 21.67}$ kips

The unfactored dead load reactions are listed below:

Unfactored Girder Reactions (kips)		
Girder #	DC	DW
1	263.0	21.7
2	305.8	21.7
3	305.8	21.7
4	305.8	21.7
5	305.8	21.7
6	263.0	21.7

Table E13-2.2-1
Unfactored Girder Dead Load Reactions

E13-2.2.2 Live Load Reactions per Design Lane

From girder line analysis, the following pier unfactored live load reactions are obtained:

$TruckPair := 125.64$ kips per design lane

$Lane := 103.94$ kips per design lane

$DLA := 1.33$ dynamic load allowance

These loads are per design lane and do not include dynamic load allowance. The pier reactions are controlled by the 90% (Truck Pair + Lane) loading condition. The resulting combined live load reactions per design lane (including dynamic load allowance) are:

$$R\_LL_{DesLane} := 0.90 \cdot (TruckPair \cdot DLA + Lane) \qquad \boxed{R\_LL_{DesLane} = 243.94} \qquad \text{kips}$$

The resulting wheel loads are:

$$R_{LLw} := \frac{0.90 \cdot TruckPair \cdot DLA}{2} \qquad \boxed{R_{LLw} = 75.2} \qquad \text{kips per wheel}$$

$$R_{LLlane} := \frac{0.90 \cdot Lane}{10} \qquad \boxed{R_{LLlane} = 9.35} \qquad \text{kips per foot}$$

E13-2.2.3 Superstructure Live Load Reactions

For the pier in this design example, the maximum live load effects in the pier cap, column and footing are based on either one, two or three lanes loaded (whichever results in the worst force effect). The lanes are moved across the deck to create the envelope of force effects. The following figures illustrate the lane locations loaded to determine the maximum positive and negative moments as well as the maximum shear force effects in the pier cap.
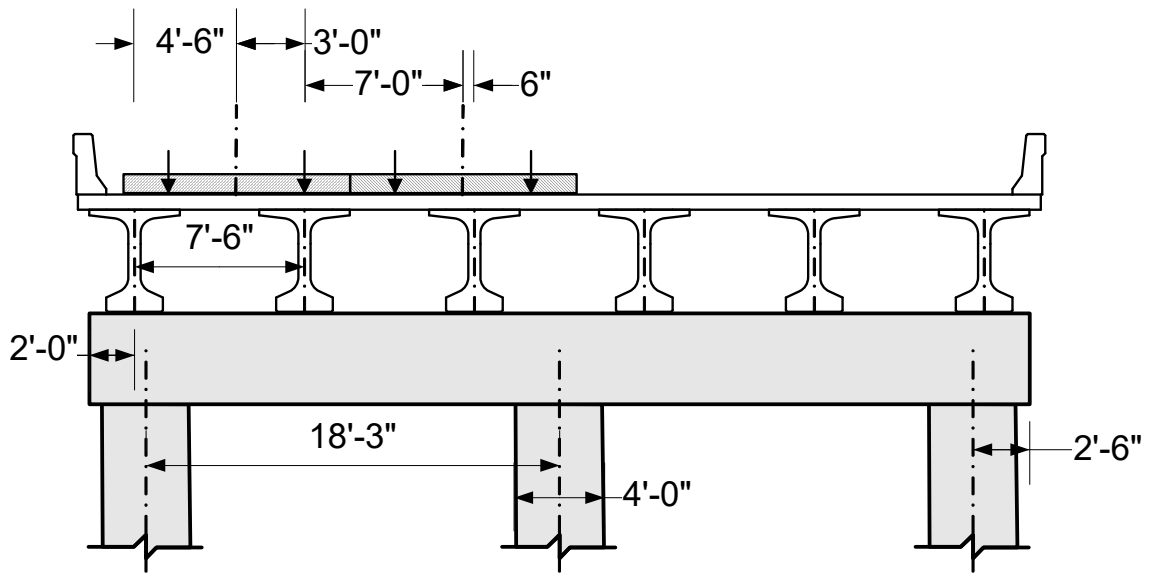
Figure E13-2.2-1
Lane Locations for Maximum Positive Moment
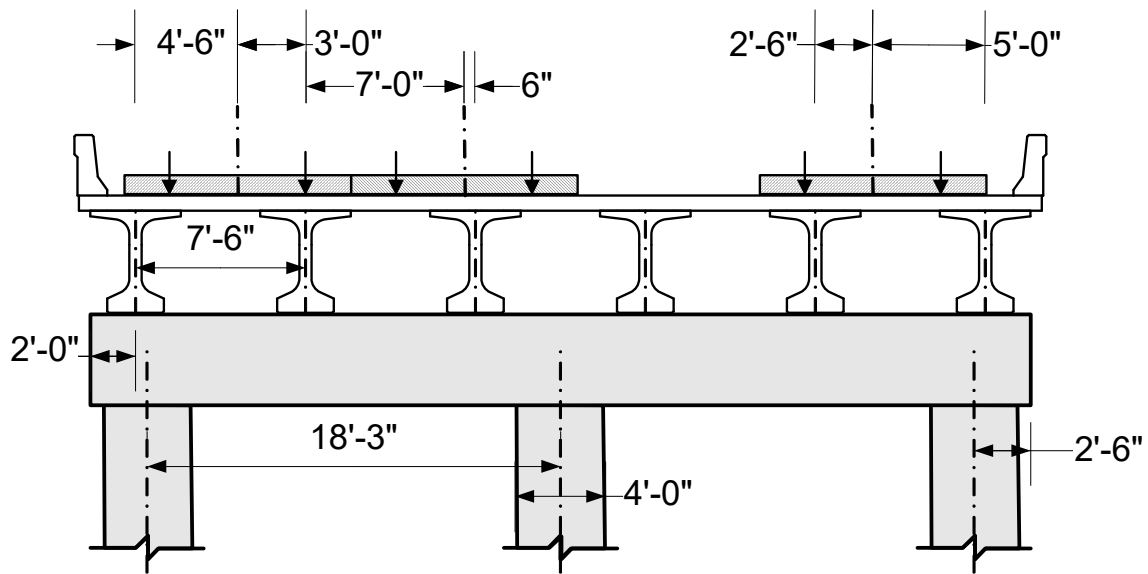
Figure E13-2.2-2
Lane Locations for Maximum Negative Moment
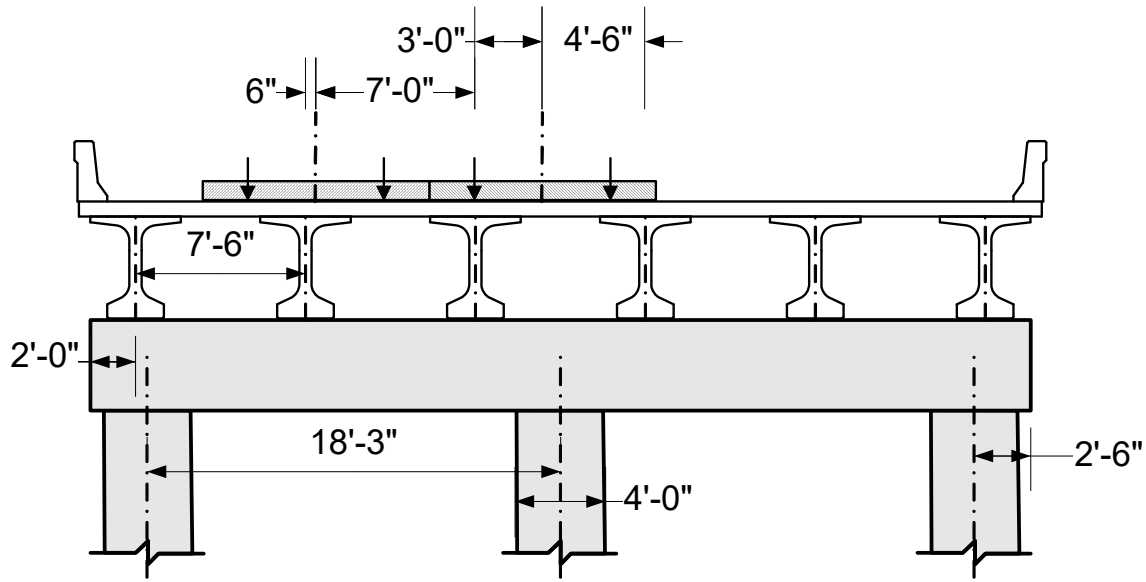
Figure E13-2.2-3
Lane Locations for Maximum Shear

The next step is to compute the reactions due to the above loads at each of the six bearing locations. This is generally carried out by assuming the deck is pinned (i.e., discontinuous) at the interior girder locations but continuous over the exterior girders. Solving for the reactions is then elementary. The computations for the reactions for maximum moment with only 2 lanes loaded are illustrated below as an example. All reactions shown are in kips.

$m_2 := 1.0$ Multi-presence factor for two lanes loaded

$$R1_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{6.0}{7.5} \right) + R_{LLlane} \cdot \left(0.5 + \frac{7.5}{2} \right) \right]$$ $\boxed{R1_{LL} = 99.91}$

$$R2_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{1.5}{7.5} + 1 + \frac{3.5}{7.5} \right) + R_{LLlane} \cdot (7.5) \right]$$ $\boxed{R2_{LL} = 195.49}$

$$R3_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{4.0 + 5.0}{7.5} \right) + R_{LLlane} \cdot \left(\frac{7.5}{2} + 4.5 \cdot \frac{5.25}{7.5} \right) \right]$$ $\boxed{R3_{LL} = 154.78}$

$$R4_{LL} := m_2 \cdot \left[R_{LLw} \cdot \left(\frac{2.5}{7.5} \right) + R_{LLlane} \cdot 4.5 \cdot \frac{2.25}{7.5} \right]$$ $\boxed{R4_{LL} = 37.69}$

$$R5_{LL} := 0$$ $\boxed{R5_{LL} = 0}$

$$R6_{LL} := 0$$ $\boxed{R6_{LL} = 0}$

E13-2.3 Unfactored Force Effects

The resulting unfactored force effects for the load cases shown above are shown in the table below. Note that the maximum shear and negative moment values are taken at the face of the column.

Unfactored Force Effects			
Effect	DC	DW	LL
Maximum Positive Moment	943.1	62.17	628.4
Maximum Negative Moment	-585.6	-39.03	-218.9
Maximum Shear	429.2	28.53	228.3
(Corresponding Moment)	-585.6	-39.03	-119.3

Table E13-2.3-1
Unfactored Force Effects

E13-2.4 Load Factors

From **LRFD [Table 3.4.1-1]**:

DC   DW   LL

$\gamma st_{DC} := 1.25$ $\gamma st_{DW} := 1.50$ $\gamma st_{LL} := 1.75$

$\gamma s1_{DC} := 1.0$ $\gamma s1_{DW} := 1.0$ $\gamma s1_{LL} := 1.0$

E13-2.5 Combined Force Effects

The resulting factored Service and Strength force effects for the load cases previously illustrated are shown in the tables below. The full Service and Strength factored moment and shear envelopes are shown in the following graphs.

Factored Service Force Effects				
Effect	DC	DW	LL	Total
Maximum Positive Moment	943.1	62.2	628.4	1633.7
Maximum Negative Moment	-585.6	-39.0	-218.9	-843.5
Maximum Shear	429.2	28.5	228.3	686.0
(Corresponding Moment)	-585.6	-39.0	-119.3	-743.9

Table E13-2.5-1
Factored Service Force Effects

Factored Strength Force Effects				
Effect	DC	DW	LL	Total
Maximum Positive Moment	1178.9	93.3	1099.7	2371.8
Maximum Negative Moment	-732.0	-58.5	-383.1	-1173.6
Maximum Shear	536.5	42.8	399.5	978.8
(Corresponding Moment)	-732.0	-58.5	-208.8	-999.3

Table E13-2.5-2
Factored Strength I Force Effects

Factored Service Moments

Distance Along Cap (ft)

Factored Service Shear

Distance Along Cap (ft)

Factored Strength Moments

Distance Along Cap (ft)

Factored Strength 1 Shear

Distance Along Cap (ft)
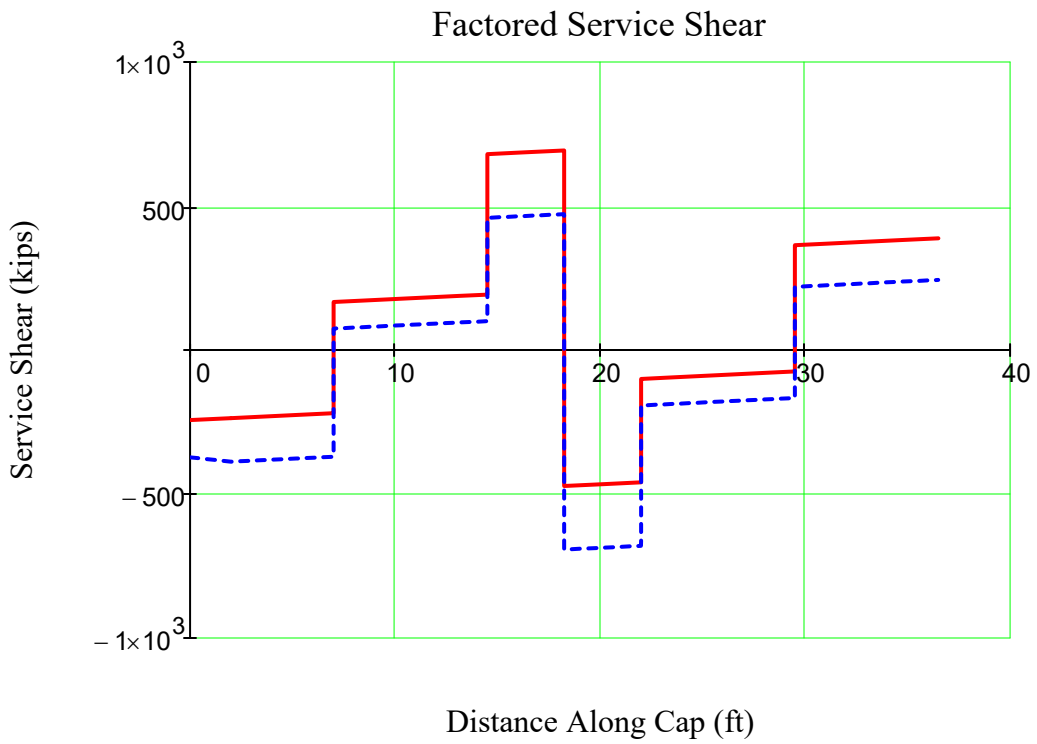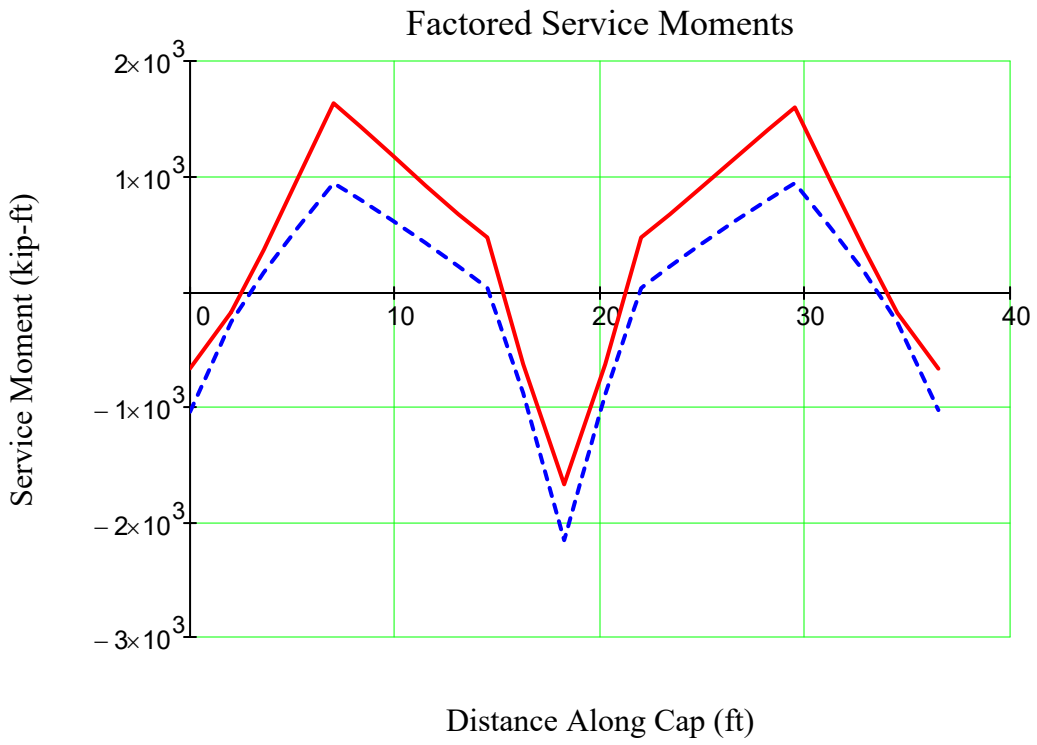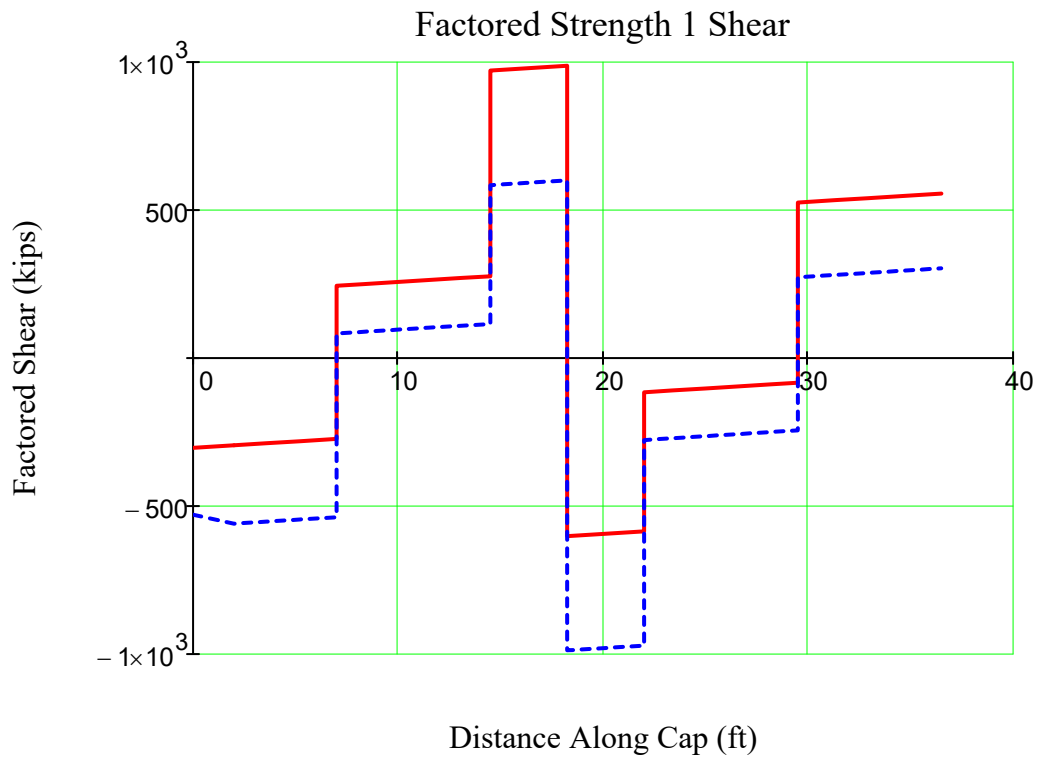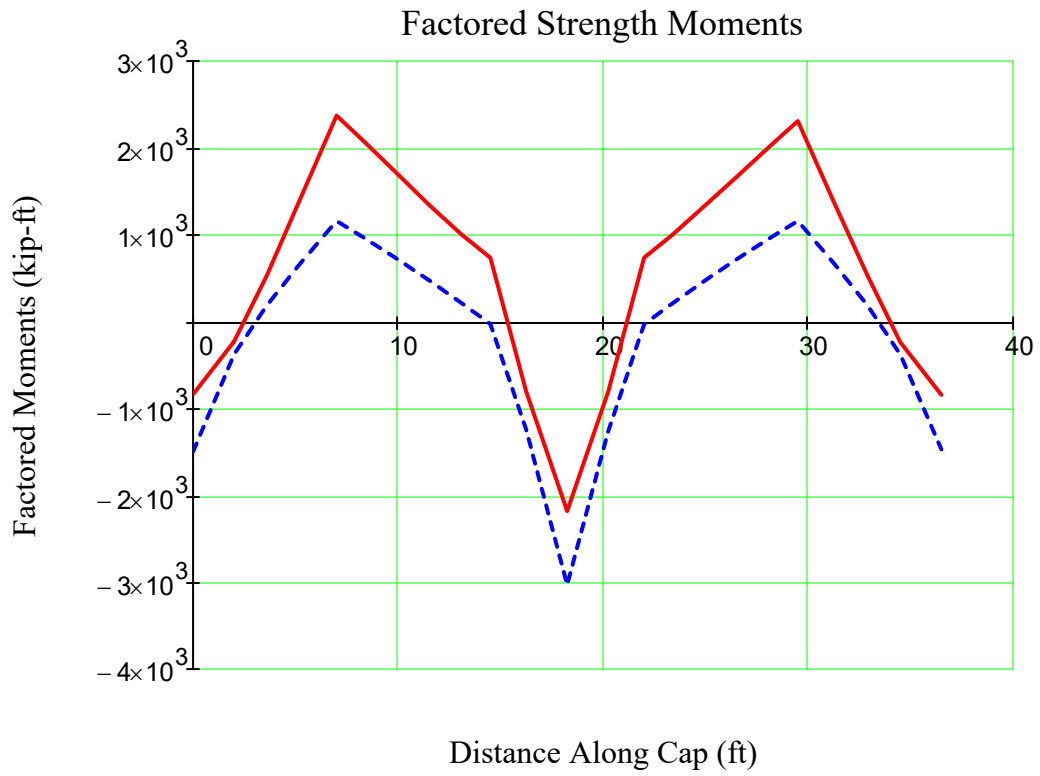
E13-2.6 Pier Cap Design

Calculate positive and negative moment requirements.

E13-2.6.1 Positive Moment Capacity Between Columns

It is assumed that there will be two layers of positive moment reinforcement. Therefore the effective depth of the section at the pier is:

$\text{cover} := 2.5$ in

In accordance with **LRFD [5.10.3.1.3]** the minimum clear space between the bars in layers is one inch or the nominal diameter of the bars.

$\text{spa}_{\text{clear}} := 1.75$ in

$\text{bar}_{\text{stirrup}} := 5$ (transverse bar size)

$\boxed{\text{Bar}_D(\text{bar}_{\text{stirrup}}) = 0.63}$ in (transverse bar diameter)

$\text{Bar}_{\text{No\_pos}} := 9$

$\boxed{\text{Bar}_D(\text{Bar}_{\text{No\_pos}}) = 1.13}$ in   (Assumed bar size)

$d_e := \text{cap}_H \cdot 12 - \text{cover} - \text{Bar}_D(\text{bar}_{\text{stirrup}}) - \text{Bar}_D(\text{Bar}_{\text{No\_pos}}) - \dfrac{\text{spa}_{\text{clear}}}{2}$

$\boxed{d_e = 42.87}$ in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.
The width of the cap:

$b_w := \text{cap}_W \cdot 12$ $\boxed{b_w = 42}$ in

$\boxed{Mu_{\text{pos}} = 2372}$ kip-ft

$R_u := \dfrac{Mu_{\text{pos}} \cdot 12}{\phi_f \cdot b_w \cdot d_e^{\,2}}$ $\boxed{R_u = 0.4097}$ ksi

$\rho := 0.85 \dfrac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \dfrac{2 \cdot R_u}{0.85 \cdot f'_c}} \right)$ $\boxed{\rho = 0.00738}$

$A_s := \rho \cdot b_w \cdot d_e$ $\boxed{A_s = 13.28}$ in²

This requires $n_{\text{bars\_pos}} := 14$ bars. Use $n_{\text{bars\_pos1}} := 9$ bars in the bottom layer and $n_{\text{bars\_pos2}} := 5$ bars in the top layer. Check spacing requirements.

$\text{spa}_{\text{pos}} := \dfrac{b_w - 2 \cdot (\text{cover} + \text{Bar}_D(\text{bar}_{\text{stirrup}})) - \text{Bar}_D(\text{Bar}_{\text{No\_pos}})}{n_{\text{bars\_pos1}} - 1}$

$\boxed{\text{spa}_{\text{pos}} = 4.33}$ in

$clear_{spa} := spa_{pos} - Bar_D(Bar_{No\_pos})$ $\boxed{clear_{spa} = 3.2}$ in

The minimum clear spacing is equal to 1.5 times the maximum aggregate size of 1.5 inches.

$spa_{min} := 1.5 \cdot 1.5$ $\boxed{spa_{min} = 2.25}$ in

Is $spa_{min} \le clear_{spa}$? $\boxed{check = \text{"OK"}}$

$As_{prov\_pos} := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos}$ $\boxed{As_{prov\_pos} = 14}$ in²

LRFD [5.6.2.2] $\alpha_1 := 0.85$ (for f'$_c \le$ 10.0 ksi)

$a := \dfrac{As_{prov\_pos} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$ $\boxed{a = 6.72}$ in

$Mn_{pos} := As_{prov\_pos} \cdot f_y \cdot \left(d_e - \dfrac{a}{2} \right) \cdot \dfrac{1}{12}$ $\boxed{Mn_{pos} = 2766}$ kip-ft

$Mr_{pos} := \phi_f \cdot Mn_{pos}$ $\boxed{Mr_{pos} = 2489}$ kip-ft

                                           $\boxed{Mu_{pos} = 2372}$ kip-ft

Is $Mu_{pos} \le Mr_{pos}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.6.3.3]**:

$S_{cap} := \dfrac{(cap_W \cdot 12) \cdot (cap_H \cdot 12)^2}{6}$ $\boxed{S_{cap} = 16128}$ in³

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]** $\boxed{f_r = 0.45}$ ksi

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_{cap}$ therefore, $M_{cr} = 1.1(f_r)S_{cap}$

Where:

     $\gamma_1 := 1.6$ flexural cracking variability factor

     $\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement
                                     <u>for A615, Grade 60 reinforcement</u>

$M_{cr} := 1.1 \cdot f_r \cdot S_{cap} \cdot \dfrac{1}{12}$ $\boxed{M_{cr} = 664}$ kip-ft

                                         $\boxed{1.33 \cdot Mu_{pos} = 3155}$ kip-ft

Is Mr_{pos} greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{pos}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.6.7]**:

$$\rho := \frac{As_{prov\_pos}}{b_w \cdot d_e}$$

$$\boxed{\rho = 0.00778}$$

$$n := floor\left(\frac{E_s}{E_c}\right)$$

$$\boxed{n = 8}$$

$$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$$

$$\boxed{k = 0.3}$$

$$j := 1 - \frac{k}{3}$$

$$\boxed{j = 0.9}$$

$$d_c := cover + Bar_D(bar_{stirrup}) + \frac{Bar_D(Bar_{No\_pos})}{2}$$

$$\boxed{d_c = 3.69} \qquad in$$

$$\boxed{Ms_{pos} = 1634} \qquad kip\text{-}ft$$

$$f_s := \frac{Ms_{pos}}{As_{prov\_pos} \cdot j \cdot d_e} \cdot 12 \quad \leq 0.6\, f_y$$

$$\boxed{f_s = 36.24} \quad ksi \quad approx. = 0.6\, f_y \ O.K.$$

The height of the section, h, is:

$$h := cap_H \cdot 12$$

$$\boxed{h = 48} \qquad in$$

$$\beta := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$$\boxed{\beta = 1.12}$$

$$\gamma_e := 1.0 \qquad\qquad \text{for Class 1 exposure condition}$$

$$S_{max} := \frac{700\gamma_e}{\beta \cdot f_s} - 2 \cdot d_c$$

$$\boxed{S_{max} = 9.89} \qquad in$$

$$\boxed{spa_{pos} = 4.33} \qquad in$$

Is $spa_{pos} \leq S_{max}$?

$$\boxed{check = "OK"}$$

E13-2.6.2 Positive Moment Reinforcement Cut Off Location

Terminate the top row of bars where bottom row of reinforcement satisfies the moment diagram.

$$spa' := spa_{pos}$$

$$\boxed{spa' = 4.33} \qquad in$$

$$As' := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos1}$$

$$\boxed{As' = 9} \qquad in^2$$

LRFD [5.6.2.2] $\boxed{\alpha_1 = 0.85}$ (for f'$_c$ ≤ 10.0 ksi)

$a' := \dfrac{As' \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$ $\boxed{a' = 4.32}$ in

$d_{e'} := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - \dfrac{Bar_D(Bar_{No\_pos})}{2}$

                                       $\boxed{d_{e'} = 44.31}$ in

$M_{n'} := As' \cdot f_y \cdot \left(d_{e'} - \dfrac{a'}{2}\right) \cdot \dfrac{1}{12}$ $\boxed{M_{n'} = 1897}$ kip-ft

$M_{r'} := \phi_f \cdot M_{n'}$ $\boxed{M_{r'} = 1707}$ kip-ft

Based on the moment diagram, try locating the first cut off at $\boxed{cut_{pos} := 10.7}$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

                                         $\boxed{M_{r'} = 1707}$ kip-ft

                                         $\boxed{Mu_{cut1} = 1538}$ kip-ft

                                         $\boxed{Ms_{cut1} = 1051}$ kip-ft

Is $Mu_{cut1} \le M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.6.3.3]**:

                                         $\boxed{M_{cr} = 664}$ kip-ft

                                         $\boxed{1.33 \cdot Mu_{cut1} = 2045}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{cut1}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.6.7]**:

$\rho' := \dfrac{As'}{b_w \cdot d_{e'}}$ $\boxed{\rho' = 0.00484}$

$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n$ $\boxed{k' = 0.24}$

$$j' := 1 - \frac{k'}{3}$$

$\boxed{j' = 0.92}$

$\boxed{Ms_{cut1} = 1051}$ kip-ft

$$f_{s'} := \frac{Ms_{cut1}}{As' \cdot j' \cdot d_{e'}} \cdot 12 \quad \leq 0.6\, f_y$$

$\boxed{f_{s'} = 34.39}$ ksi $\leq 0.6\, f_y$ O.K.

$\boxed{\beta = 1.12}$

$\boxed{\gamma_e = 1}$

$$S_{max'} := \frac{700\gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c$$

$\boxed{S_{max'} = 10.81}$ in

$\boxed{spa' = 4.33}$ in

Is $spa' \leq S_{max'}$?

$\boxed{check = \text{"OK"}}$

The bars shall be extended past this cut off point for a distance not less than the following, **LRFD [5.10.8.1.2a]**:

$\boxed{d_{e'} = 44.31}$ in

$\boxed{15 \cdot Bar_D(Bar_{No\_pos}) = 16.92}$ in

$\boxed{\dfrac{col_{spa} \cdot 12}{20} = 10.95}$ in

$\boxed{BarExtend_{pos} = 44.31}$ in

The bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number

$\boxed{\quad\rightarrow 9\quad}$ bar with spacing less than 6-inches, is:

$l_{d\_9} := 5.083$ ft

$\boxed{cut_{pos} + \dfrac{BarExtend_{pos}}{12} = 14.39}$

$\boxed{0.4 \cdot col_{spa} + l_{d\_9} = 12.38}$

Similar calculations show that the second layer bottom mat bars can also be terminated at a distance of 2.0 feet from the CL of the left column. At least one quarter of the bars shall be

extended past the centerline of the support for continuous spans. Therefore, run the bottom layer bars to the end of the cap.

E13-2.6.3 Negative Moment Capacity at Face of Column

It is assumed that there will be one layer of negative moment reinforcement. Therefore the effective depth of the section at the pier is:

$$\text{cover} = 2.5 \qquad \text{in}$$

$$\text{bar}_{\text{stirrup}} = 5 \qquad \text{(transverse bar size)}$$

$$\text{Bar}_D(\text{bar}_{\text{stirrup}}) = 0.63 \qquad \text{in (transverse bar diameter)}$$

$$\text{Bar}_{\text{No\_neg}} := 8$$

$$\text{Bar}_D(\text{Bar}_{\text{No\_neg}}) = 1.00 \qquad \text{in} \quad \text{(Assumed bar size)}$$

$$d_{\text{e\_neg}} := \text{cap}_H \cdot 12 - \text{cover} - \text{Bar}_D(\text{bar}_{\text{stirrup}}) - \frac{\text{Bar}_D(\text{Bar}_{\text{No\_neg}})}{2}$$

$$d_{\text{e\_neg}} = 44.38 \qquad \text{in}$$

For flexure in non-prestressed concrete, $\boxed{\phi_f = 0.9}$.
The width of the cap:

$$b_w = 42 \qquad \text{in}$$

$$Mu_{\text{neg}} = -1174 \qquad \text{kip-ft}$$

$$R_{\text{u\_neg}} := \frac{|Mu_{\text{neg}}| \cdot 12}{\phi_f \cdot b_w \cdot d_{\text{e\_neg}}^2} \qquad\qquad R_{\text{u\_neg}} = 0.1892 \qquad \text{ksi}$$

$$\rho_{\text{neg}} := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_{\text{u\_neg}}}{0.85 \cdot f'_c}}\right) \qquad\qquad \rho_{\text{neg}} = 0.00326$$

$$A_{\text{s\_neg}} := \rho_{\text{neg}} \cdot b_w \cdot d_{\text{e\_neg}} \qquad\qquad A_{\text{s\_neg}} = 6.08 \qquad \text{in}^2$$

This requires $n_{\text{bars\_neg}} := 9$ bars. Check spacing requirements.

$$\text{spa}_{\text{neg}} := \frac{b_w - 2 \cdot (\text{cover} + \text{Bar}_D(\text{bar}_{\text{stirrup}})) - \text{Bar}_D(\text{Bar}_{\text{No\_neg}})}{n_{\text{bars\_neg}} - 1}$$

$$\text{spa}_{\text{neg}} = 4.34 \qquad \text{in}$$

$$clear_{spa\_neg} := spa_{neg} - Bar_D(Bar_{No\_neg})$$

$$\boxed{clear_{spa\_neg} = 3.34} \quad \text{in}$$

Is $spa_{min} \leq clear_{spa\_neg}$? $\qquad\qquad\qquad\qquad$ $\boxed{check = \text{"OK"}}$

$$As_{prov\_neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg}$$

$$\boxed{As_{prov\_neg} = 7.07} \quad \text{in}^2$$

LRFD [5.6.2.2] $\qquad\qquad \alpha_1 := 0.85 \qquad$ (for f'$_c \leq$ 10.0 ksi)

$$a_{neg} := \frac{As_{prov\_neg} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$

$$\boxed{a_{neg} = 3.39} \qquad\qquad \text{in}$$

$$Mn_{neg} := As_{prov\_neg} \cdot f_y \cdot \left(d_{e\_neg} - \frac{a_{neg}}{2}\right) \cdot \frac{1}{12}$$

$$\boxed{Mn_{neg} = 1508} \qquad \text{kip-ft}$$

$$Mr_{neg} := \phi_f \cdot Mn_{neg}$$

$$\boxed{Mr_{neg} = 1358} \qquad \text{kip-ft}$$

$$\boxed{Mu_{neg} = 1174} \qquad \text{kip-ft}$$

Is $Mu_{neg} \leq Mr_{neg}$? $\qquad\qquad\qquad\qquad$ $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.6.3.3]**:

$$\boxed{M_{cr} = 664} \qquad \text{kip-ft}$$

$$\boxed{1.33 \cdot Mu_{neg} = 1561} \qquad \text{kip-ft}$$

Is Mr_{neg} greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{neg}$? $\qquad$ $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.6.7]**:

$$\rho_{neg} := \frac{As_{prov\_neg}}{b_w \cdot d_{e\_neg}}$$

$$\boxed{\rho_{neg} = 0.00379}$$

$$\boxed{n = 8}$$

$$k_{neg} := \sqrt{(\rho_{neg} \cdot n)^2 + 2 \cdot \rho_{neg} \cdot n} - \rho_{neg} \cdot n$$

$$\boxed{k_{neg} = 0.22}$$

$$j_{neg} := 1 - \frac{k_{neg}}{3}$$

$$\boxed{j_{neg} = 0.93}$$

$$d_{c\_neg} := cover + Bar_D(bar_{stirrup}) + \frac{Bar_D(Bar_{No\_neg})}{2}$$

$$\boxed{d_{c\_neg} = 3.63} \qquad \text{in}$$

$$\boxed{Ms_{neg} = 844} \qquad \text{kip-ft}$$

$$f_{s\_neg} := \frac{Ms_{neg}}{As_{prov\_neg} \cdot j_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \leq 0.6\,f_y \qquad f_{s\_neg} = 34.8 \quad \text{ksi} \quad \leq 0.6\,f_y\,\text{O.K.}$$

The height of the section, h, is:

$h = 48$ in

$$\beta_{neg} := 1 + \frac{d_{c\_neg}}{0.7 \cdot (h - d_{c\_neg})} \qquad\qquad \beta_{neg} = 1.12$$

$\gamma_e := 1.0$ for Class 1 exposure condition

$$S_{max\_neg} := \frac{700\gamma_e}{\beta_{neg} \cdot f_{s\_neg}} - 2 \cdot d_{c\_neg} \qquad\qquad S_{max\_neg} = 10.76 \quad \text{in}$$

$spa_{neg} = 4.34$ in

Is $spa_{neg} \leq S_{max\_neg}$?   check = "OK"

E13-2.6.4 Negative Moment Reinforcement Cut Off Location

Cut 4 bars where the remaining 5 bars satisfy the moment diagram.

$n_{bars\_neg'} := 5$

$spa'_{neg} := spa_{neg} \cdot 2$ $spa'_{neg} = 8.69$ in

$As'_{neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg'}$ $As'_{neg} = 3.93$ in^2

LRFD [5.6.2.2] $\alpha_1 := 0.85$ (for f'$_c \leq$ 10.0 ksi)

$$a'_{neg} := \frac{As'_{neg} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c} \qquad\qquad a'_{neg} = 1.89 \quad \text{in}$$

$d_{e\_neg} = 44.38$ in

$$M_{n'\_neg} := As'_{neg} \cdot f_y \cdot \left(d_{e\_neg} - \frac{a'_{neg}}{2}\right) \cdot \frac{1}{12} \qquad M_{n'\_neg} = 853 \quad \text{kip-ft}$$

$$M_{r'\_neg} := \phi_f \cdot M_{n'\_neg} \qquad\qquad M_{r'\_neg} = 768 \quad \text{kip-ft}$$

Based on the moment diagram, try locating the cut off at $cut_{neg} := 15.3$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$$M_{r'\_neg} = 768 \qquad \text{kip-ft}$$

$$Mu_{neg\_cut} = 577 \qquad \text{kip-ft}$$

$$Ms_{neg\_cut} = 381 \qquad \text{kip-ft}$$

Is $Mu_{neg\_cut} \leq M_{r'\_neg}$?

$$\boxed{check = \text{"OK"}}$$

Check the minimum reinforcement limits in accordance with **LRFD [5.6.3.3]**:

$$M_{cr} = 664 \qquad \text{kip-ft}$$

$$1.33 \cdot Mu_{neg\_cut} = 767 \quad \text{kip-ft}$$

Is $M_{r'\_neg}$ greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{neg\_cut}$?

$$\boxed{check = \text{"OK"}}$$

Check the Service I crack control requirements in accordance with **LRFD [5.6.7]**:

$$\rho'_{neg} := \frac{As'_{neg}}{b_w \cdot d_{e\_neg}}$$

$$\boxed{\rho'_{neg} = 0.00211}$$

$$k'_{neg} := \sqrt{\left(\rho'_{neg} \cdot n\right)^2 + 2 \cdot \rho'_{neg} \cdot n} - \rho'_{neg} \cdot n$$

$$\boxed{k'_{neg} = 0.17}$$

$$j'_{neg} := 1 - \frac{k'_{neg}}{3}$$

$$\boxed{j'_{neg} = 0.94}$$

$$Ms_{neg\_cut} = 381 \qquad \text{kip-ft}$$

$$f_{s'\_neg} := \frac{Ms_{neg\_cut}}{As'_{neg} \cdot j'_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \leq 0.6\, f_y$$

$$\boxed{f_{s'\_neg} = 27.79} \quad \text{ksi} \quad \leq 0.6\, f_y \text{ O.K.}$$

$$\boxed{\beta_{neg} = 1.12}$$
$$\boxed{\gamma_e = 1}$$

$$S_{max'\_neg} := \frac{700 \gamma_e}{\beta_{neg} \cdot f_{s'\_neg}} - 2 \cdot d_{c\_neg}$$

$$\boxed{S_{max'\_neg} = 15.30} \qquad \text{in}$$

$$\boxed{spa'_{neg} = 8.69} \qquad \text{in}$$

Is $spa'_{neg} \leq S_{max'\_neg}$?

$$\boxed{check = \text{"OK"}}$$

The bars shall be extended past this cut off point for a distance not less than the following,
LRFD [5.10.8.1.2c]:

$d_{e\_neg} = 44.38$ in

$12 \cdot Bar_D(Bar_{No\_neg}) = 12$ in

$\dfrac{(col_{spa} - col_w) \cdot 12}{16} = 10.69$ in

$BarExtend_{neg} = 44.38$ in

These bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number

$\rightarrow 8$ "top" bar with spacing greater than 6-inches, is:

$l_{d\_8} := 3.25$ ft

The cut off location is determined by the following:

$cut_{neg} - \dfrac{BarExtend_{neg}}{12} = 11.6$ ft

$col_{spa} - \dfrac{col_w}{2} - l_{d\_8} = 13$ ft

Therefore, the cut off location is located at the following distance from the CL of the left column:

$cutoff_{location} = 11.6$ ft

By inspection, the remaining top mat reinforcement is adequate over the exterior columns. The inside face of the exterior column is located at:

$col_{face} := \dfrac{col_w}{2} \cdot \dfrac{1}{col_{spa}}$ $col_{face} = 0.11$ % along cap

$Mu_{negative}(col_{face}) = -378.37$ kip-ft

$Ms_{negative}(col_{face}) = -229.74$ kip-ft

E13-2.6.5 Shear Capacity at Face of Center Column

$\boxed{Vu = 978.82}$ kips

The Factored Shear Resistance, V_r

$V_r = \phi_v(V_n)$

$\phi_v := 0.9$

V_n is determined as the lesser of the following equations, **LRFD [5.7.3.3]**:

$V_{n1} = V_c + V_s + V_p$

$V_{n2} = 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$

V_c , the shear resistance due to concrete (kip), is calculated as follows:

$V_c = 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$

Where:
         b_v = effective web width (in) taken as the minimum section width within the depth d_v
         d_v = effective shear depth (in), the distance, measured perpendicular to the neutral
           axis between the resultants of the tensile and compressive force due to flexure.
           It need not be taken less than the greater of $0.9d_e$ or $0.72h$

$b_v := cap_W \cdot 12$ $\boxed{b_v = 42}$ in

                                                $\boxed{d_{e\_neg} = 44.38}$ in

                                                  $\boxed{a_{neg} = 3.39}$ in

$d_{v\_neg} := d_{e\_neg} - \dfrac{a_{neg}}{2}$ $\boxed{d_{v\_neg} = 42.68}$ in

                                               $\boxed{0.9 \cdot d_{e\_neg} = 39.94}$ in

                        $\boxed{h = 48}$ in $\boxed{0.72 \cdot h = 34.56}$ in

Therefore, use $\boxed{d_v = 42.68}$ in for V_c calculation.

$\beta := 2.0$ Factor indicating ability of diagonally cracked concrete to transmit tension.
                 For nonprestressed sections, $\beta = 2.0$, **LRFD [5.7.3.4.1]**.

$\lambda := 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$V_c := 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_c = 211.94}$ kips

V_s , the shear resistance due to steel (kips), is calculated as follows:

$V_s = \dfrac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s}$

Where:

    s = spacing of stirrups (in)

    θ = angle of inclination of diagonal compressive stresses (deg)

    α = angle of inclination of transverse reinforcement to longitudinal axis (deg)

$s := 5$ in

$\theta := 45\,deg$ for non prestress members

$\alpha := 90\,deg$ for vertical stirrups

A_v = (# of stirrup legs)(area of stirrup)

$bar_{stirrup} = 5$

$StirrupConfig := \text{"Triple"}$

$stirrup_{legs} = 6$

$A_v := stirrup_{legs} \cdot \left(Bar_A \left(bar_{stirrup} \right) \right)$ $A_v = 1.84$ in²

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s}$$ $V_s = 942.74$ kips

V_p , the component of the effective prestressing force in the direction of the applied shear:

$V_p := 0$ for non prestressed members

V_n is the lesser of:

$V_{n1} := V_c + V_s + V_p$ $V_{n1} = 1154.67$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $V_{n2} = 1568.41$ kips

Therefore, use: $V_n = 1154.67$ kips

$V_r := \phi_v \cdot V_n$ $V_r = 1039.2$ kips

     $Vu = 978.82$ kips

Is $Vu \leq V_r$? $check = \text{"OK"}$

Check the Minimum Transverse Reinforcement, **LRFD [5.7.2.5]**

Required area of transverse steel:

$\lambda := 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

$Av_{min} := 0.0316 \cdot \dfrac{\lambda \sqrt{f'_c} \cdot b_v \cdot s}{f_y}$

$\boxed{Av_{min} = 0.21}$ in²

$\boxed{A_v = 1.84}$ in²

Is $Av_{min} \leq A_v$ (provided area of steel)? $\boxed{\text{check} = \text{"OK"}}$

Check the Maximum Spacing of the Transverse Reinforcement, **LRFD [5.7.2.6]**

If $v_u < 0.125f'_c$, then: $s_{max} := 0.8 \cdot d_v \leq 24$ in

If $v_u >$ or $= 0.125f'_c$, then: $s_{max} := 0.4 \cdot d_v \leq 12$ in

The shear stress on the concrete, v_u, is taken to be:

$v_u := \dfrac{Vu}{\phi_v \cdot b_v \cdot d_v}$

$\boxed{v_u = 0.61}$ ksi

$\boxed{0.125 \cdot f'_c = 0.44}$ ksi

$\boxed{s_{max} = 12}$ in

$\boxed{s = 5}$ in

Is the spacing provided $s \leq s_{max}$? $\boxed{\text{check} = \text{"OK"}}$

Similar calculations are used to determine the required stirrup spacing for the remainder of the cap.

$\boxed{s_2 = 12}$ in $\boxed{s_3 = 6}$ in

$\boxed{\text{StirrupConfig}_2 = \text{"Double"}}$ $\boxed{\text{StirrupConfig}_3 = \text{"Double"}}$

$\boxed{Vu2 = 276}$ kips $\boxed{Vu3 = 560}$ kips

$\boxed{V_{r\_2} = 408.94}$ kips $\boxed{V_{r\_3} = 627.13}$ kips

It should be noted that the required stirrup spacing is typically provided for a distance equal to the cap depth past the CL of the girder. Consideration should also be given to minimize the number of stirrup spacing changes where practical. These procedures result in additional capacity in the pier cap that is often beneficial for potential future rehabilitation work on the structure.

E13-2.6.6 Temperature and Shrinkage Steel

Temperature and shrinkage steel shall be provided on each face and in each direction as calculated below. **LRFD [5.10.6]**

$$cap_W = 3.5 \qquad \text{ft}$$

$$cap_H = 4 \qquad \text{ft}$$

$$b := cap_W \cdot 12 \qquad\qquad b = 42 \qquad \text{in}$$

$$h = 48 \qquad \text{in}$$

$$As_{ts} := \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} \qquad\qquad As_{ts} = 0.24 \qquad \text{in}^2\text{/ft in each face}$$

Is the area required As_{ts} between 0.11 and 0.60 in^2 per foot? $check = \text{"OK"}$

Use number 5 bars at one foot spacing: $Bar_A(5) = 0.31 \qquad \text{in}^2\text{/ft in each face}$

E13-2.6.7 Skin Reinforcement

If the effective depth, d_e, of the reinforced concrete member exceeds 3 ft., longitudinal skin reinforcement is uniformly distributed along both side faces of the component for a distance of $d_e/2$ nearest the flexural tension reinforcement, **LRFD [5.6.7]**. The area of skin reinforcement (in^2/ft of height) on each side of the face is required to satisfy:

$$A_{sk} \geq 0.012(d_e - 30) \quad \text{and} \quad A_{sk} \cdot \left(\frac{d_e}{2 \cdot 12}\right) \quad \text{need not exceed } (A_s / 4)$$

Where: (For positive moment region)

A_{sk} = area of skin reinforcement (in^2/ft)

A_s = area of tensile reinforcement (in^2) $A_s = 13.28 \qquad \text{in}^2$

d_e = flexural depth taken as the distance from the compression face to the centroid of the steel, positive moment region (in) $d_e = 42.87 \qquad \text{in}$

$$A_{sk1} := 0.012 \cdot (d_e - 30) \qquad\qquad A_{sk1} = 0.15 \qquad \text{in}^2\text{/ft}$$

$$A_{sk1} := A_{sk1} \cdot \left(\frac{d_e}{2 \cdot 12}\right) \qquad\qquad A_{sk1} = 0.28 \qquad \text{in}^2$$

$$A_{sk2} := \frac{A_s}{4} \qquad\qquad A_{sk2} = 3.32 \qquad \text{in}^2$$

$$A_{face} := \min(A_{sk1}, A_{sk2}) \quad \begin{array}{l}\text{(area req'd. per face}\\ \text{within } d_e/2 \text{ from tension}\\ \text{reinf.)}\end{array} \qquad A_{face} = 0.28 \qquad \text{in}^2$$

$$spa\_max_{sk} := \min\left(\frac{d_e}{6}, 12\right) \qquad\qquad spa\_max_{sk} = 7.15 \qquad \text{in}$$

Use number 5 bars at 6" spacing: $Bar_A(5) \cdot 2 = 0.61 \qquad \text{in}^2 \quad > A_{face}$
(provides 2 bars within $d_e/2$ from tension reinf.)

Preceding calculations looked at skin reinforcement requirements in the positive moment region. For the negative moment region, #5 bars at 6" will also meet its requirements.

E13-2.7 Reinforcement Summary
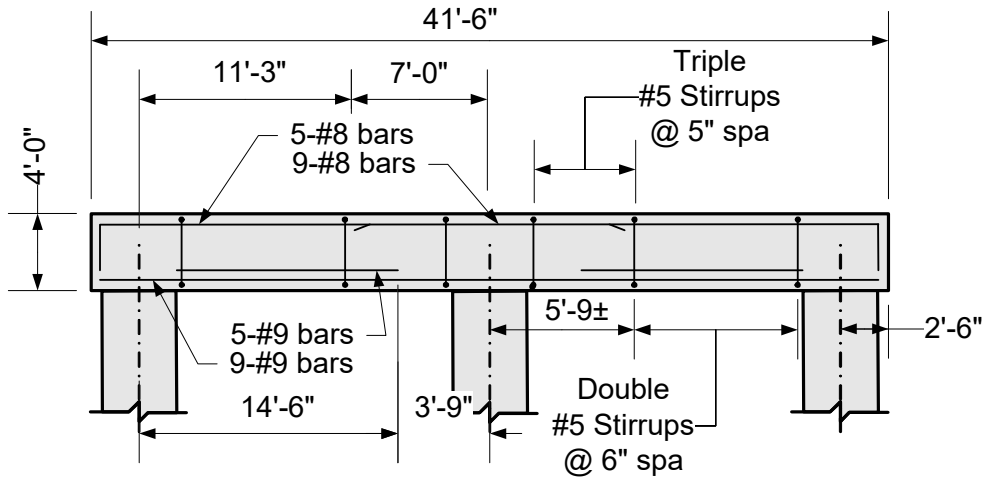
Figure E13-2.7-1
Cap Reinforcement - Elevation View
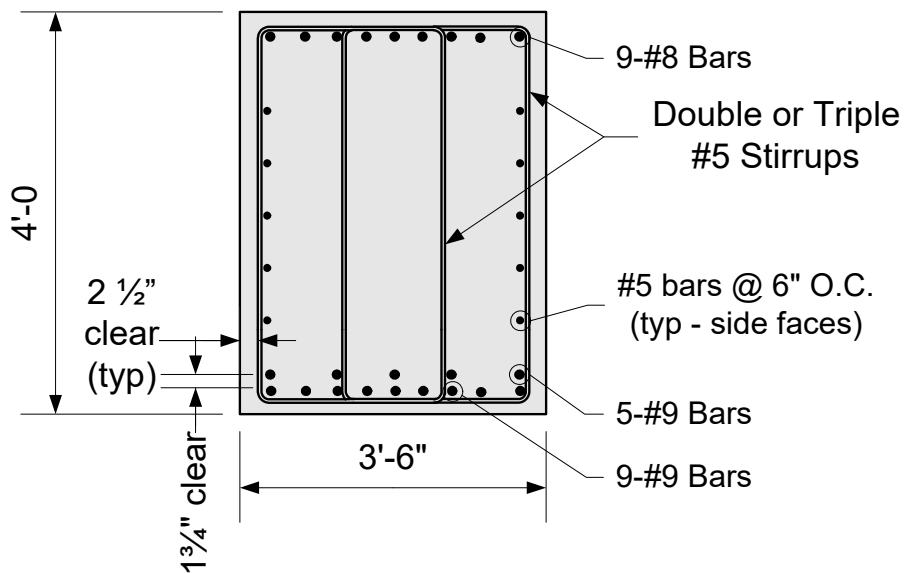
Figure E13-2.7-2
Cap Reinforcement - Section View

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a slight over estimation of K_a, therefore, increasing the pressure on the wall resulting in a more conservative design.

Walls that are cast-in-place (CIP) semi gravity concrete cantilever referred, hereafter, as CIP cantilever, Mechanically Stabilized Earth (MSE), modular block gravity, soil nailing, soldier-pile and sheet-pile walls are typically considered flexible enough to justify using an active earth pressure coefficient.

For walls using Coulomb earth pressure theory:

$$K_a = \frac{\sin^2(\theta + \phi_f')}{\Gamma[\sin^2\theta \sin(\theta - \delta)]}$$ **LRFD [Eq'n 3.11.5.3-1]**

Where:

$$\Gamma \quad = \quad \left[1 + \sqrt{\frac{\sin(\phi_f' + \delta)\sin(\phi_f' - B)}{\sin(\theta - \delta)\sin(\theta + B)}}\right]^2$$

δ =   Friction angle between fill and wall (degrees)

B =   Angle of fill to the horizontal (degrees)

θ =   Angle of back face of wall to the horizontal (degrees)

ϕ_f' =   Effective angle of internal friction (degrees)

Note: refer to Figure 14.4-1 for details.

For walls using Rankine earth pressure theory:

$$K_a = \tan^2\left(45 - \frac{\phi_f'}{2}\right)$$

At-Rest Earth Pressure

In the at-rest earth pressure (K_o) condition, the top of the wall is not allowed to deflect or rotate; therefore, requiring the wall to support the full pressure of the soil behind the wall.

The at-rest earth pressure coefficient shall be used to calculate the lateral earth pressure for non-yielding retaining walls restrained from rotation and/or lateral translation in accordance with **LRFD [3.11.5.2]**. Non-yielding walls include integral abutment walls, or retaining walls resting on bedrock or pile foundation.

For walls (normally consolidated soils, vertical wall, and level ground) using at-rest earth pressure:

$$K_o = 1 - \sin\ \phi_f'$$ **LRFD [Eq'n 3.11.5.2-1]**

Passive Earth Pressure

The development of passive earth pressure (K_p) requires a retaining wall to move into or toward the soil. As with the active earth pressure, Rankine earth pressure is the preferred method to be used to develop passive earth pressure coefficient. The use of Rankine theory will cause an under estimation of K_p, therefore resulting in a more conservative design. Coulomb earth pressure theory may be used if the appropriate conditions exist at a site; however, the designer is required to understand the limitations on the use of Coulomb earth pressure theory as applied to passive earth pressures.

Neglect any contribution from passive earth pressure in stability calculations unless the base of the wall extends below the depth to which foundation soil or rock could be weakened or removed by freeze-thaw, shrink-swell, scour, erosion, construction excavation, or any other means. In wall stability calculations, only the embedment below this depth, known as the effective embedment depth, shall be considered when calculating the passive earth pressure resistance. This is in accordance with **LRFD [11.6.3.5].**

14.4.5.4.1 Earth Load Surcharge

The effect of earth load surcharge including uniform, strip, and point loads shall be computed in accordance with **LRFD [3.11.6.1]** and **LRFD [3.11.6.2].**

14.4.5.4.2 Live Load Surcharge

Increased earth pressure on a wall occurs due to vehicular loading on top of the retained earth including operation of large or heavily-loaded cranes, staged equipment, soil stockpile or material storage, or any surcharge loads behind the walls. Earth pressure from live load surcharge shall be applied when a vehicular load is within one half of the wall height behind the back face of the wall or reinforced soil mass for MSE walls, in accordance with **LRFD [3.11.6.4].** In most cases, surcharge load can be modeled by assuming 2 ft of fill.

WisDOT policy item:

The equivalent height of soils for vehicular loading on retaining walls parallel to the traffic shall be 2.0 feet, regardless of the wall height. For standard unit weight of soil equal to 120 pcf, the resulting live load surcharge is 240 psf. Walls without traffic shall be designed for a live load surcharge of 100 psf to account for construction live loads.

14.4.5.4.3 Compaction Loads

Pressure induced by the compaction load can extend to variable depths due to the total static and dynamic forces exerted by compaction equipment. The effect of increased lateral earth pressure due to compaction loads during construction should be considered when compaction equipment is operated behind the wall. The compaction load surcharge effect is minimized by WISDOT standard specifications that require small walk behind compactors within 3 ft of the wall.
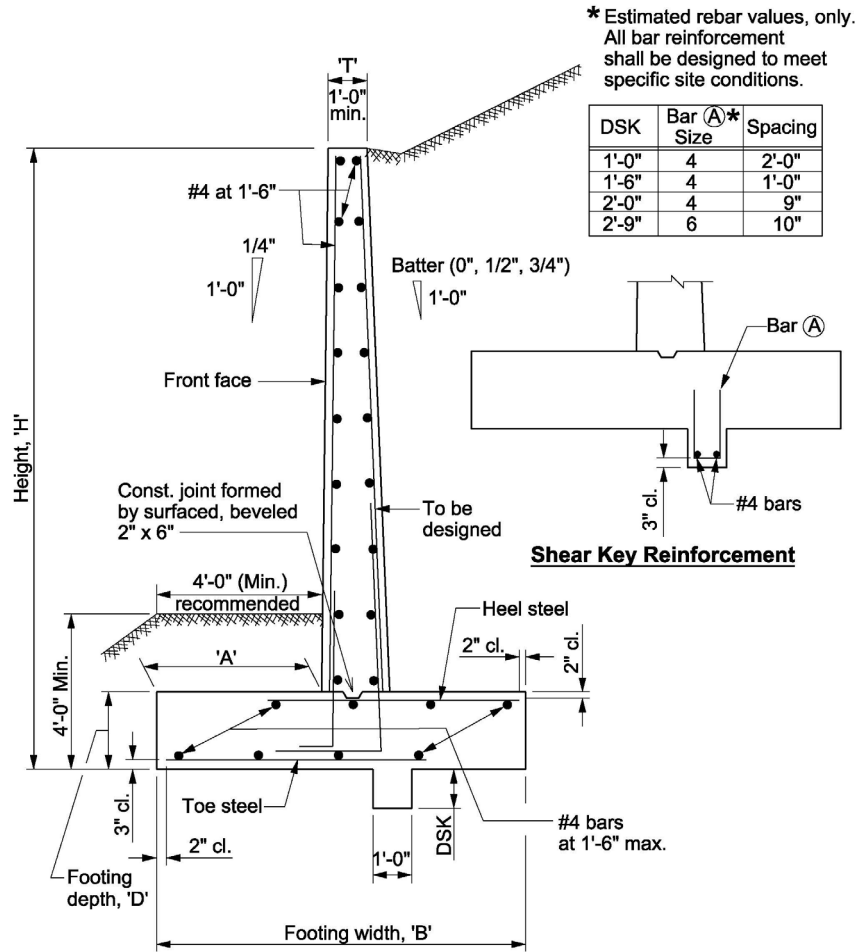
Figure 14.5-2
CIP Walls General Details

14.5.3.1 Wall Back and Front Slopes

CIP walls shall not be designed for backfill slope steeper than 2:1(H:V). Where practical, walls shall have a horizontal bench of 4.0 feet wide at the front face.

14.5.4 Unfactored and Factored Loads

Unfactored loads and moments are computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure 14.4-1 for the earth pressure is developed assuming a triangular distribution plus additional pressures resulting from earth surcharge, water pressure, compaction or any other loads, etc. The material

properties for backfill soil, concrete and steel are given in 14.4.6. The foundation and retained earth properties as recommended in the Geotechnical Report shall be used for computing nominal loads.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. Figure 14.4-8 shows load factor and load combinations along with their application for the load limit state evaluation. A summary of load factors and load combinations as applicable for a typical CIP cantilever wall is presented in Table 14.4-1 and **LRFD [3.4.1]**, respectively. Computed factored loads and moments are used for performing stability checks.

14.5.5 External Stability Checks

The external stability check includes checks for limiting eccentricity (overturning), bearing stress, and sliding at Strength I and Extreme Event II due to vehicle impact in cases where live load traffic is carried.

14.5.5.1 Eccentricity Check

The eccentricity of the retaining wall shall be evaluated in accordance with **LRFD [11.6.3.3]**. The location of the resultant force should be within 1/3 of base width of the foundation centroid (e<B/3) for foundations on soil, and within 0.45 of the base width of the foundation centroid (e<0.45B) for foundations on rock. If there is inadequate resistance to overturning (eccentricity value greater than limits given above), consideration should be given to either increasing the width of the wall base, or providing a deep foundation.

14.5.5.2 Bearing Resistance

The bearing resistance shall be evaluated at the strength limit state using factored loads and resistances. Bearing resistance of the walls founded directly on soil or rock shall be computed in accordance with 11.2 and **LRFD [10.6]**. The bearing resistance for walls on piles shall be computed in accordance with 11.3 and **LRFD [10.6]**. Figure 14.5-3 shows bearing stress criteria for a typical CIP wall on soil and rock respectively.

The vertical stress for footings on soil shall be calculated using:

$$\sigma_v = \frac{\sum V}{(B-2e)}$$

For walls founded on rock, the vertical stress is calculated assuming a linearly distributed pressure over an effective base area. The vertical stress for footings on rock shall be computed using:

$$\sigma_v = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right)$$

Retained Soil

- Unit weight = 120 lb/ft^3

- Angle of internal friction =

  o Use value provided by Geotechnical Engineer

- Rock-infill unit weight =

  o Based on porosity and rock type

Soil Pressure Theory

- Coulomb's Theory for prefabricated wall systems

- Rankine theory or Coulomb theory, at the discretion of designer for gabion walls

7   Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{ES}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50	1.50	Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	1.50	Bearing, wall strength
Service I	1.00	1.00	1.00	1.00	1.00		Global, settlement, wall crack control

Table 14.8-1
Load Factor Summary for Prefabricated Modular Walls

14.9 Soil Nail Walls

Soil nail walls consist of installing reinforcement of the ground behind an excavation face, by drilling and installing closely-spaced rows of grouted steel bars (i.e., soil nails). The soil nails are grouted in place and subsequently covered with a facing; used to stabilize the exposed excavation face, support the sub-drainage system (i.e., composite strip drain, collector and drainage pipes), and distribute the soil nail bearing plate load over a larger area. When used for permanent applications, a permanent facing layer, meeting the aesthetic and structural requirement is constructed directly over the temporary facing.

Soil nail walls are typically used to stabilize excavation during construction. Soil nail walls have been used recently with MSE walls to form hybrid wall systems typically known as 'shored walls'. The soil nails are installed as top down construction. Conventional soil nail wall systems are best suited for sites with dense to very dense, granular soil with some apparent cohesion (sands and gravels), stiff to hard, fine-grained soil (silts and clays) of relatively low plasticity (PI<15), or weak, weathered massive rock with no adversely-oriented planes of weakness. Soil nail wall construction requires that open excavations stand unsupported long enough to allow soil nail drilling and grouting, sub-drainage installation, reinforcement, and temporary shotcrete placement. Soil nail walls should not be used below groundwater.

14.9.1 Design Requirements

The design of soil nail walls shall be in accordance with **LRFD [11.12]**.The FHWA publication FHWA-NHI-14-007 (*Geotechnical Engineering Circular No. 7 – Soil Nail Walls-Reference Manual)* is the recommended design manual for soil nail walls. The design life of the soil nail walls shall be in accordance with 14.4.3.

The design of the soil nailing walls requires an evaluation of external, internal, and overall stability and facing-connection failure modes as presented in publication FHWA-NHI-14-007.

A permanent wall facing is required for all permanent soil nail walls. Permanent facing is commonly constructed of cast-in-place (CIP) concrete, welded wire mesh (WWM) reinforced concrete and precast fabricated panels. In addition to meeting the aesthetic requirements and providing adequate corrosion protections to the soil nails, design facings for all facing-connection failure modes indicated in publication FHWA-NHI-14-007.

Corrosion protection is required for all permanent soil nail wall systems to assure adequate long-term wall durability. The level of corrosion protection required should be determined on a project-specific basis based on factors such as wall design life, structure criticality and the electrochemical properties of the supporting soil and rock materials. Criteria for classification of the supporting soil and rock materials as "aggressive" or "non-aggressive" are provided in publication FHWA-NHI-14-007.

Soil nails are field tested to verify that nail design loads can be supported without excessive movement and with an adequate margin of safety. Perform both verification and proof testing of designated test nails as recommended in publication FHWA-NHI-14-007.

14.17 References

1. State of Wisconsin, Department of Transportation, *Facilities Development Manual*

2. American Association of State highway and Transportation officials. *Standard Specification for highway Bridges*

3. American Association of State highway and Transportation officials. *AASHTO LRFD Bridge Design Specifications*

4. AASHTO LRFD Bridge Design Specification 4th Edition, 2007, AASHTO, 444 North Capitol Street, N.W., Suite 249, Washington, D.C. 20001.

5. Berg, Christopher and Samtani. *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*. Publication No. FHWA-NHI-10-024.2009.

6. Bowles, Joseph E. *Foundation Analysis and Design 4th Edition.* McGraw Hill *1989*

7. Cudoto, Donald P. *Foundation Design Principles and Practices (* 2nd Edition*),* Prentice Halls

8. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls", 2302 Horse Pen Road, Herndon, Virginia 22071-3406.

9. Publication No. FHWA-NHI-14-007, "Geotechnical Engineering Circular No. 7 Soil Nail Walls – Reference Manual"

10. Publication No. FHWA-SA-96-069R, "Manual for design and construction of Soil Nail walls"

11. Publication No. FHWA-HI-98-032, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures"

12. Publication No. FHWA-NHI-07-071, "Earth retaining Structures"

13. Publication No. FHWA-NHI-09-083, "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures"

14. Publication No. FHWA-NHI-09-087, "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced slopes"

15. Publication No. FHWA-NHI-10-024, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume I"

16. Publication No. FHWA-NHI-10-025, "Design and Construction of Mechanically Stabilized earth Walls and Reinforced Soil Slopes-Volume II"

14.18 Design Examples

E14-1   Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD

E14-2   Precast Panel Steel Reinforced MSE Wall, LRFD

E14-3   Modular Block Facing Geogrid Reinforced MSE Wall, LRFD

E14-4   Cast-In-Place Concrete Cantilever Wall on Piles, LRFD

E14-5   Sheet Pile Wall, LRFD

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and/or bicycle traffic, a live load of 90 psf is used. Consideration should also be given to maintenance vehicle loads as specified in Chapter 37 – Pedestrian Bridges.

17.2.5 Load Factors

The load factor, γ_i, is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis and the probability of simultaneous occurrence of different loads.

For the design limit states, the values of γ_i for different types of loads are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. Load factors most commonly used for superstructure design are also presented in Table 17.2-5.

Load Combination	Load Factor, γ_i				
	DC		DW		LL+IM
	Maximum	Minimum	Maximum	Minimum	
Strength I	1.25	0.90	1.50	0.65	1.75
Strength III	1.25	0.90	1.50	0.65	0.00
Strength V	1.25	0.90	1.50	0.65	1.35
Service I	1.00	1.00	1.00	1.00	1.00
Service II	1.00	1.00	1.00	1.00	1.30
Service III	1.00	1.00	1.00	1.00	0.80
Fatigue I	0.00	0.00	0.00	0.00	1.75
Extreme Event II	1.00	1.00	1.00	1.00	0.50

Table 17.2-5
Load Factors

The maximum and minimum values should be used to maximize the intended effect of the load. An example of the use of minimum load factors is the load factor for dead load when uplift is being checked.

17.2.6 Resistance Factors

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

Resistance factors are presented in **LRFD [1.3.2.1]**, **LRFD [5.5.4.2]**, **LRFD [6.5.4.2]**, **LRFD [6.5.5]** and **LRFD [6.10.1.7]**. The most commonly used resistance factors for superstructure design are also presented in Table 17.2-6.

Limit State	Material	Application	Resistance Factor, ϕ
Strength	Concrete	Flexure (reinforced concrete)	0.90
		Flexure (prestressed concrete)	1.00
		Shear (normal weight)	0.90
		Shear (lightweight)	0.90
	Steel	Flexure	1.00
		Shear	1.00
		Axial compression, steel only	0.95
		Axial compression, composite	0.90
		Tension, fracture in net section	0.80
		Tension, yielding in gross section	0.95
		Bolts bearing on material	0.80
		Shear connectors	0.85
		A325 and A490 bolts in tension	0.80
		A325 and A490 bolts in shear	0.80
		A307 bolts in tension	0.80
		A307 bolts in shear	0.75
		Block shear	0.80
		Web crippling	0.80
		Welds	See **LRFD [6.5.4.2]**
Service	All	All	1.0
Fatigue	All	All	1.0
Extreme Event	All	All	1.0

Table 17.2-6
Resistance Factors

17.2.7 Distribution of Loads for Slab Structures

For slab structures, the distribution of loads is based on strip widths, as illustrated in Figure 17.2-6 through Figure 17.2-11. Figure 17.2-6 and Figure 17.2-7 illustrate the distribution of loads for slab structures with no sidewalks. Figure 17.2-8 and Figure 17.2-9 illustrate the distribution of loads for slab structures with sidewalks. Figure 17.2-10 and Figure 17.2-11 illustrate the distribution of loads for slab structures with raised sidewalks. It should be noted that, although medians are not shown in these figures, medians are treated similar to other superimposed dead loads.

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18.1 Introduction

18.1.1 General

This chapter considers the following types of concrete structures:

- Flat Slab

- Haunched Slab

A longitudinal slab is one of the least complex types of bridge superstructures. It is composed of a single element superstructure in comparison to the two elements of the transverse slab on girders or the three elements of a longitudinal slab on floor beams supported by girders. Due to simplicity of design and construction, the concrete slab structure is relatively economical. Its limitation lies in the practical range of span lengths and maximum skews for its application. For longer span applications, the dead load becomes too high for continued economy. Application of the haunched slab has increased the practical range of span lengths for concrete slab structures.

18.1.2 Limitations

Concrete slab structure types are not recommended over streams where the normal water freeboard is less than 4 feet; formwork removal requires this clearance. When spans exceed 35 feet, freeboard shall be increased to 5 feet above normal water.

All concrete slab structures are limited to a maximum skew of 30 degrees. Slab structures with skews in excess of 30 degrees, require analysis of complex boundary conditions that exceed the capabilities of the present design approach used in the Bureau of Structures.

Continuous span slabs are to be designed using the following pier types:

- Piers with pier caps (on columns or shafts)

- Wall type piers

These types will allow for ease of future superstructure replacement. Piers that have columns without pier caps, have had the columns damaged during superstructure removal. This type of pier will not be allowed without the approval of the Structures Design Section.

WisDOT policy item:

Slab bridges, due to camber required to address future creep deflection, do not ride ideally for the first few years of their service life and present potential issues due to ponding. As such, if practical (e.g. not excessive financial implications), consideration of other structure types should be given for higher volume/higher speed facilities, such as the Interstate. Understanding these issues, the Regions have the responsibility to make the final decision on structure type with respect to overall project cost, with BOS available for consultation.

18.2 Specifications, Material Properties and Structure Type

18.2.1 Specifications

Reference may be made to the design and construction related material as presented in the following specifications:

- *State of Wisconsin, Department of Transportation Standard Specifications for Highway and Structure Construction*

  Section 502 - Concrete Bridges

  Section 505 - Steel Reinforcement

- Other Specifications as referenced in Chapter 3

18.2.2 Material Properties

The properties of materials used for concrete slab structures are as follows:

f'_c =   specified compressive strength of concrete at 28 days, based on cylinder tests

4 ksi, for concrete slab superstructure

3.5 ksi, for concrete substructure units

f_y =   60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s =   29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c =   modulus of elasticity of concrete in slab **LRFD [C5.4.2.4]**

= $33{,}000\ K_1\ w_c^{1.5}\ (f'_c)^{1/2} = 3800$ ksi

Where:

K_1 =   1.0

w_c =   0.150 kcf, unit weight of concrete

n   = $E_s / E_c = 8$ **LRFD [5.6.1]**   (modular ratio)

18.2.3 Structure Type and Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, approximate slab depth, skew, roadway width, etc.. The selection of the type of concrete slab structure

(haunched / flat) is a function of the span lengths selected. Recommended span length ranges and corresponding structure type are shown for single-span and multiple-span slabs in Figure 18.2-1. Estimated slab depths are shown in Table 18.2-1.

Currently, voided slab structures are not allowed. Some of the existing voided slabs have displayed excessive longitudinal cracking over the voids in the negative zone. This may have been caused by the voids deforming or floating-up due to lateral pressure during the concrete pour. Recent research indicates slabs with steel void-formers have large crack widths above the voids due to higher stress concentrations.

If optimum span ratios are selected such that the positive moments in each span are equal, the interior and end span slab depths will be equal, provided Strength Limit State controls. Optimum span ratios are independent of applied live loading.

SINGLE SPAN
Span Length < 50 feet: Use Flat Slab

TWO OR MORE SPANS

All Span Lengths < 35 feet ± : Use Flat Slab Throughout
Any Span Length ≥ 45 feet ± < 70 feet: Use Haunched Slab Throughout
Other Spans: Consider Economics, Aesthetics and Clearances
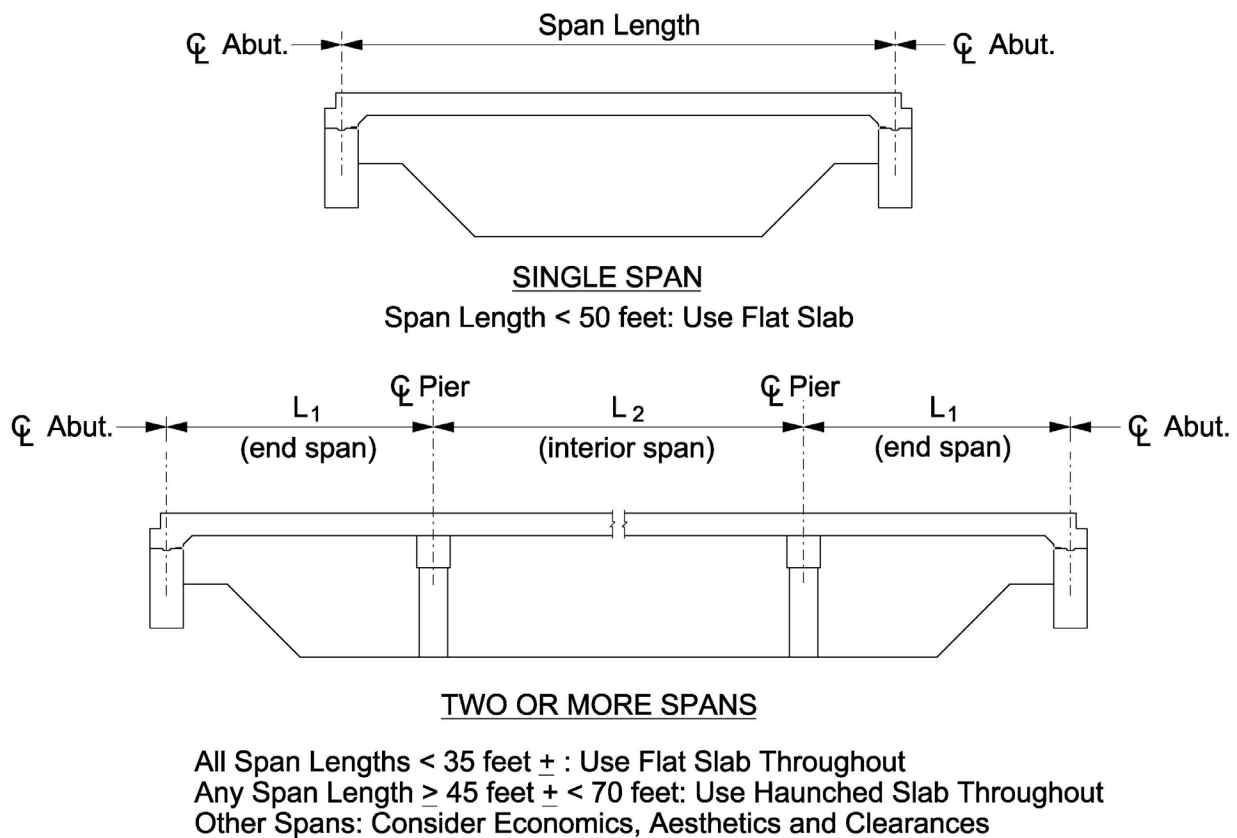
Figure 18.2-1
Span Length vs. Slab Type

For the following optimum span ratio equations based on Strength Limit State controlling, L_1 equals the end span lengths and L_2 equals the interior span length or lengths, for structures with three or more spans.

For flat slabs the optimum span ratio is obtained when $L_2 = 1.25L_1$. The optimum span ratio for a three-span haunched slab results when $L_2 = L_1(1.43 - 0.002L_1)$ and for a four-span haunched slab when $L_2 = 1.39L_1$.

Approximate slab depths for multiple-span flat and haunched slabs can be obtained from Table 18.2-1. These values are to be used for dead load computations and preliminary computations only and the final slab depth is to be determined by the designer.

(s) Span Length (feet)	Slab Depth (inches)	
	Haunched [1]	Flat [4]
20	---	12
25	---	14
30	---	16
35	---	18
40	---	20
45	16 [2]	22
50	17.5 [2]	24
55	19 [2]	26
60	20 [2]	---
65	22 [3]	---
70	25 [3]	---

Table 18.2-1
Span Length vs. Slab Depth

[1] These estimated slab depths at mid-span, apply to interior spans of three or more span structures, with an end span length of approximately 0.7 times the interior span. Depths are based on dead load deflection (camber) and live load deflection limits. Haunch length (L_{haunch}) = 0.167 (L_2), and d_{slab} / D_{haunch} = 0.6 were used. L_2 = interior span length, (d_{slab}) = slab depth in span and (D_{haunch}) = slab depth at haunch. Values in table include ½ inch wearing surface.

[2] Depths controlled by live load deflection criteria

[3] Depths controlled by dead load deflection (camber) criteria

[4] These values represent **LRFD [2.5.2.6.3]** recommended minimum depths for continuous-spans using (s+10)/30. The slab span length (s) in the equation and resulting minimum depths are in feet and are presented in inches in Table 18.2-1. For simple-spans, the Bureau of Structures adds 10% greater depth and checks the criteria in 18.4.4. Values in table include ½ inch wearing surface.

The minimum slab depth is 12 inches. Use increments of ½ inch to select depths > 12 inches.

18.3 Limit States Design Method

18.3.1 Design and Rating Requirements

All new concrete slab structures are to meet design requirements as stated in 17.1.1 and rating requirements as stated in 17.1.2.

18.3.2 LRFD Requirements

18.3.2.1 General

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: **LRFD [1.3.2.1, 5.5.1]**

$$Q = \sum \eta_i\, \gamma_i\, Q_i \leq \phi\, R_n = R_r \qquad \text{(Limit States Equation)} \qquad \textbf{LRFD [1.3.2.1, 3.4.1]}$$

Where:

η_i	=	load modifier (a function of η_D , η_R and η_I) **LRFD [1.3.2.1, 1.3.3, 1.3.4, 1.3.5]**
γ_i	=	load factor
Q_i	=	force effect; moment, shear, stress range or deformation caused by applied loads
Q	=	total factored force effect
ϕ	=	resistance factor
R_n	=	nominal resistance; resistance of a component to force effects
R_r	=	factored resistance = $\phi\, R_n$

The Limit States used for concrete slab design are:

- Strength I Limit State

- Service I Limit State

- Fatigue I Limit State

18.3.2.2 Statewide Policy

Current Bureau of Structures policy is :

- Set value of load modifier, η_i , and its factors (η_D , η_R , η_I) all equal to 1.00 for concrete slab design.

- Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.21]** that would reduce force effects, Q_i , for slab bridges.

- Ignore reduction factor, r, for skewed slab bridges in **LRFD [4.6.2.3]** that would reduce longitudinal force effects, Q_i .

18.3.3 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a bridge is expected to experience during its design life **LRFD [1.3.2.4]**. The total factored force effect, Q, must not exceed the factored resistance, R_r, as shown in the equation in 18.3.2.1.

Strength I Limit State **LRFD [3.4.1]** will be used for:

- Designing longitudinal slab reinforcement for flexure

- Designing transverse slab reinforcement over the piers for flexure

- Checking shear (two-way) in slab at the piers

- Checking uplift at the abutments

- Checking longitudinal slab reinforcement for tension from shear

18.3.3.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Strength I Limit State will be used to design the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, defined in 18.4.3.2.

The load factor, γ_i , is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

For Strength I Limit State, the values of γ_i for each applied load, are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]** and their values are: γ_{DC} = 1.25/0.90, γ_{DW} = 1.50/0.65, γ_{LL+IM} = γ_{PL} = 1.75. The values for γ_{DC} and γ_{DW} have a maximum and minimum value.

Therefore, for Strength I Limit State:

   Q = 1.0 [1.25(DC) + 1.50(DW) + 1.75((LL + IM) + PL)]

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads. The load factors shown for DC and DW are maximum values. Use maximum or minimum values as shown in **LRFD [Table 3.4.1-2]** to calculate the critical force effect.

18.3.3.2 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for Strength Limit State **LRFD [5.5.4.2]** are:

- ϕ = 0.90 for flexure & tension (for tension-controlled reinforced concrete sections as defined in **LRFD [5.6.2.1]**)

- ϕ = 0.90 for shear and torsion

The factored resistance, R_r (M_r, V_r, T_{cap}), associated with the list of items to be designed/checked using Strength I Limit State in 18.3.3, are described in the following sections.

18.3.3.2.1 Moment Capacity

Stress is assumed proportional to strain below the proportional limit on the stress-strain diagram. Tests have shown that at high levels of stress in concrete, stress is not proportional to strain. Recognizing this fact, strength analysis takes into account the nonlinearity of the stress-strain diagram. This is accomplished by using a rectangular stress block to relate the concrete compressive stress distribution to the concrete strain. The compressive stress block has a uniform value of $\alpha_1 \cdot f'_c$ over a zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance a = $\beta_1 \cdot (c)$ from the extreme compression fiber. The distance (c) is measured perpendicular to the neutral axis. The factor α_1 shall be taken as 0.85 for concrete strengths not exceeding 10.0 ksi and the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi **LRFD [5.6.2.2]**. Strength predictions using this method are in agreement with strength test results. The representation of these assumptions is shown in Figure 18.3-1.

The moment capacity (factored resistance) of concrete components shall be based on the conditions of equilibrium and strain compatibility, resistance factors as specified in **LRFD [5.5.4.2]** and the assumptions outlined in **LRFD [5.6.2]**.
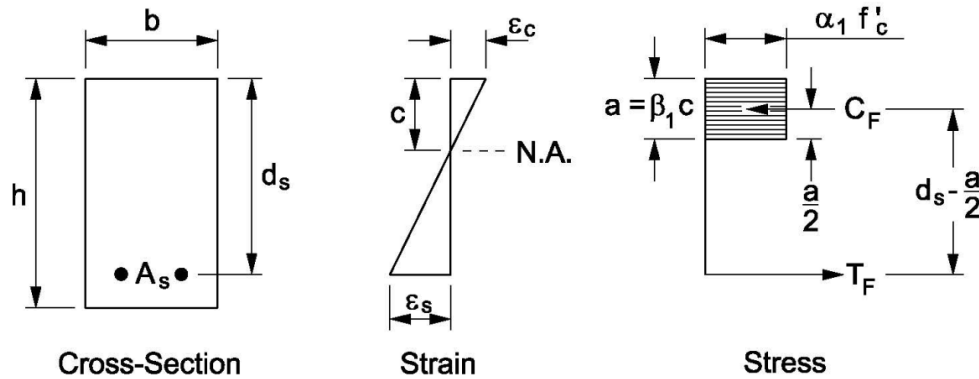
Figure 18.3-1
Stress / Strain on Cross - Section

Referring to Figure 18.3-1, the internal force equations are:

$C_F = \alpha_1 \cdot (f'_c)(b)(a) = 0.85\,(f'_c)(b)(a)$

$T_F = (A_s)(f_s)$

By equating C_F to T_F, and solving for the compressive stress block depth, (a), gives:

$a = A_s\,f_s\,/\,0.85\,(f'_c)(b)$

Use $(f_s = f_y)$ when the steel yields prior to crushing of the concrete. To check for yielding, assume $(f_s = f_y)$ and calculate the value for (a). Then calculate the value for $c = a\,/\,\beta_1$ and d_s as shown in Figure 18.3-1. If $c\,/\,d_s$ does not exceed the value calculated below, then the reinforcement has yielded and the assumption is correct, as stated in **LRFD [5.6.2.1]**.

$c\,/\,d_s \le 0.003\,/\,(0.003 + \varepsilon_{cl})$

ε_{cl} = compression controlled strain limit

for f_y = 60 ksi, ε_{cl} is 0.0020 per **LRFD [Table C5.6.2.1-1]**

if $c\,/\,d_s \le 0.6$, then the reinforcement (f_y = 60 ksi) will yield and $(f_s = f_y)$

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals: **LRFD [5.6.3.2.3]**

$M_n = A_s\,f_s\,(d_s - a/2)$

The factored resistance, M_r, or moment capacity, shall be taken as: **LRFD [5.6.3.2.1]**

$M_r = \phi\,M_n = \phi\,A_s\,f_s\,(d_s - a/2)$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$M_r = (0.9) \, A_s \, f_s \, (d_s - a/2)$

18.3.3.2.2 Shear Capacity

The nominal shear resistance, V_n , for two-way action, shall be determined as: **LRFD [5.7.1.4, 5.12.8.6.3]**

$V_n = (0.063 + 0.126 / \beta_c) \, \lambda \, (f'_c)^{\frac{1}{2}} \, b_o \, d_v \;\leq\; 0.126 \, \lambda \, (f'_c)^{\frac{1}{2}} \, b_o \, d_v$ (kips)

Where:

f'_c	=	4.0 ksi (for concrete slab bridges)
β_c	=	ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted
d_v	=	effective shear depth as determined in **LRFD [5.7.2.8]** (in)
b_o	=	perimeter of the critical section (in)
λ	=	conc. density modification factor ; for normal weight conc. = 1.0 , **LRFD [5.4.2.8]**

The factored resistance, V_r , or shear capacity, shall be taken as: **LRFD [5.7.2.1]**

$V_r = \phi \, V_n$

The resistance factor, ϕ, is 0.90, therefore:

$V_r = (0.9) \, V_n$

18.3.3.2.3 Uplift Check

The check of uplift at abutments does not use a factored resistance, but compares factored dead load and live load reactions.

18.3.3.2.4 Tensile Capacity – Longitudinal Reinforcement

The nominal tensile resistance, T_{nom} , for an area, A_s , of developed reinforcement, equals:

$T_{nom} = A_s \, f_y$

The factored resistance, T_{cap} , or tensile capacity, shall be taken as:

$T_{cap} = \phi \, T_{nom} = \phi \, A_s \, f_y$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$T_{cap} = (0.9)\, A_s\, f_y$

18.3.4 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Service I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for crack control criteria

- Checking transverse slab reinforcement over the piers for crack control criteria

- Checking live load deflection criteria

- Checking dead load deflection (camber) criteria

18.3.4.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Service I Limit State will be used to analyze the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and/or appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, where applicable, defined in 18.4.3.2.

For Service I Limit State, the values of γ_i for each applied load, are found in **LRFD [Table 3.4.1-1]** and their values are: $\gamma_{DC} = \gamma_{DW} = \gamma_{LL+IM} = \gamma_{PL} = 1.0$

Therefore, for Service I Limit State:

$Q = 1.0\,[\,1.0(DC) + 1.0(DW) + 1.0((LL + IM) + PL)\,]$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads.

18.3.4.2 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

The factored resistance, R_r , associated with the list of items to be checked using Service I Limit State in 18.3.4, are described in the following sections.

18.3.4.2.1 Crack Control Criteria

All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

18.3.4.2.2 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits. The Bureau of Structures limits live load deflections for concrete slab structures to L/1200. The deflections are based on entire slab width acting as a unit and gross moment of inertia, I_g.

The nominal resistance, R_n, or deflection limit, is:

$R_n = L/1200$

Where:

    L   =   span length

The factored resistance, R_r, is:

$R_r = \phi \, R_n = \phi \, (L/1200)$

The resistance factor, ϕ, is 1.00, therefore:

$R_r = (1.0) \, R_n = (L/1200)$

18.3.4.2.3 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g. Bureau of Structures calculates full camber based on multiplying the dead load deflection values by a factor of three. A maximum allowable camber has been set for simple-span slabs and continuous-span slabs as shown in 18.4.4.2.

The nominal resistance, R_n, or deflection limit, is:

$R_n = $ (maximum allowable camber) $/ 3$

The factored resistance, R_r, is:

$R_r = \phi \, R_n = \phi \,$ (maximum allowable camber) $/ 3$

The resistance factor, ϕ, is 1.00, therefore:

R_r = (1.0) R_n = (maximum allowable camber) / 3

18.3.5 Fatigue Limit State

Fatigue I Limit State shall be applied as a restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles **LRFD [1.3.2.3]**. The Fatigue I Limit State is intended to limit crack growth under repetitive loads to prevent fracture of the reinforcement during the design life of the bridge. The factored force effect (stress range), Q, must not exceed the factored resistance, R_r, as shown in the equation in 18.3.2.1.

For fatigue considerations, concrete members shall satisfy: **LRFD [5.5.3.1]**

$\eta_i \, \gamma_i \, (\Delta f) \leq (\Delta F)_{TH}$

Where:

γ_i	=	Load factor for Fatigue I Limit State
Δf	=	Force effect, live load stress range due to the passage of the fatigue truck (ksi)
$(\Delta F)_{TH}$	=	Constant-amplitude fatigue threshold (ksi)

Fatigue I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for fatigue stress range criteria

18.3.5.1 Factored Loads (Stress Range)

The value of the load modifier, η_i, is 1.00, as stated in 18.3.2.2.

Fatigue I Limit State will be used to analyze the structure for force effects, $Q_i = (\Delta f)$, due to applied (Fatigue Truck) live load, LL and IM, defined in 18.4.3.1.

For Fatigue I Limit State, the value of γ_i for the applied live load, is found in **LRFD [Table 3.4.1-1]** and its value is γ_{LL+IM} = 1.75.

Therefore, for Fatigue I Limit State:

Q = 1.0 [1.75(LL + IM)]

Where LL and IM represent force effects, Δf, due to these applied loads.

18.3.5.2 Factored Resistance

The resistance factor, ϕ, for Fatigue Limit State, is found in **LRFD [C1.3.2.1]** and its value is 1.00 .

18.3.5.2.1 Fatigue Stress Range

The nominal resistance, $R_n = (\Delta F)_{TH}$, for fatigue stress range (for straight reinforcement), is: **LRFD [5.5.3.2]**

$$R_n = (\Delta F)_{TH} = 26 - 22\ f_{min}\ /\ f_y \quad \text{(ksi)}$$

Where:

f_{min}	=	the minimum stress resulting from the factored Fatigue Truck live load, combined with the stress from the dead loads on the structure; positive if tension, negative if compression (ksi)
f_y	=	minimum yield strength (ksi), not to be taken less than 60 ksi nor greater than 100 ksi

The factored resistance, R_r (for f_y = 60 ksi), is:

$$R_r = \phi\ R_n = \phi\ (26 - 0.37\ f_{min})$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0)\ R_n = 26 - 0.37\ f_{min} \quad \text{(ksi)}$$

18.4 Concrete Slab Design Procedure

18.4.1 Trial Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, skew, roadway width, etc.. The selection of the type of concrete slab structure (haunched / flat) is a function of the span lengths selected. Recommended span length ranges and corresponding structure type are shown for single-span and multiple-span slabs in Figure 18.2-1. Optimum span ratios for multiple-span slabs are suggested in 18.2.3. Knowing the span lengths and the structure type, a trial slab depth can be obtained from Table 18.2-1.

For haunched slabs, the haunch depth, D_{haunch} , is proportional to the slab depth, d_{slab} , outside the haunch. A trial haunch depth can be selected as:

$$D_{haunch} = d_{slab} / 0.6$$

An economical haunch length, L_{haunch} , measured from C/L of pier to end of haunch, can be approximated between (0.15 L_2 to 0.18 L_2), where L_2 is the length of an interior span.

NOTE: With preliminary structure sizing complete, check to see if structure exceeds limitations in 18.1.2.

18.4.2 Dead Loads (DC, DW)

Dead loads (permanent loads) are defined in **LRFD [3.3.2]**. Concrete dead load is computed by using a unit weight of 150 pcf, with no adjustment in weight for the bar steel reinforcement.

DC   =   dead load of structural components and any nonstructural attachments

DW   =   dead load of future wearing surface (F.W.S.) and utilities

The slab dead load, DC_{slab} , and the section properties of the slab, do not include the ½ inch wearing surface. A post dead load, DW_{FWS} , of 20 psf, for possible future wearing surface (F.W.S.), is required in the design by the Bureau of Structures. The ½ inch wearing surface load, $DC_{1/2" WS}$, of 6 psf must also be included in the design of the slab.

Dead loads, DC, from parapets, medians and sidewalks are uniformly distributed across the full width of the slab when designing an interior strip. For the design of exterior strips (edge beams), any of these dead loads, DC, that are located directly over the exterior strip width and on the cantilevered portion of sidewalks, shall be applied to the exterior strip. For both interior and exterior strips, the future wearing surface, DW, located directly over the strip width shall be applied to it. See 17.2.7 for the distribution of dead loads.

18.4.3 Live Loads

18.4.3.1 Vehicular Live Load (LL) and Dynamic Load Allowance (IM)

The *AASHTO LRFD* Specifications contain several live load components (see 17.2.4.2) that are combined and scaled to create live load combinations that apply to different Limit States **LRFD [3.6.1]**.

The live load combinations used for design are:

LL#1:	Design Tandem (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#2:	Design Truck (+ IM) + Design Lane Load	**LRFD [3.6.1.3.1]**
LL#3:	90% [Double Design Trucks (+ IM) + Design Lane Load]	**LRFD [3.6.1.3.1]**
LL#4:	Fatigue Truck (+ IM)	**LRFD [3.6.1.4.1]**
LL#5:	Design Truck (+ IM)	**LRFD [3.6.1.3.2]**
LL#6:	25% [Design Truck (+ IM)] + Design Lane Load	**LRFD [3.6.1.3.2]**

Table 18.4-1
Live Load Combinations

The dynamic load allowance, IM, **LRFD [3.6.2]** for the live load combinations above, is shown in Table 18.4-2.

Where (IM) is required, multiply the loads by (1 + IM/100) to include the dynamic effects of the load. (IM) is not applied to the Design Lane Load.

The live load combinations are applied to the Limit States as shown in Table 18.4-2.

The live load force effect, Q_i , shall be taken as the largest from the live loads shown in Table 18.4-2 for that Limit State.

Strength I Limit State: [1]	LL#1 , LL#2 , LL#3 [2]	IM = 33%
Service I Limit State: [1] (for crack control criteria)	LL#1 , LL#2 , LL#3 [2]	IM = 33%
Service I Limit State: (for LL deflection criteria)	LL#5 , LL#6	IM = 33%
Fatigue I Limit State: [3]	LL#4 (single Fatigue Truck)	IM = 15%

Table 18.4-2
Live Loads for Limit States

[1] Load combinations shown are used for design of interior strips and exterior strips without raised sidewalks, as shown in Figures 17.2-6 to 10. For an exterior strip with a raised sidewalk,

use Design Lane Load portion of LL#2 for Live Load Case 1 and use Design Truck (+IM) portion of LL#2 for Live Load Case 2, as shown in Figure 17.2-11.

[2] (LL#3) is used to calculate negative live load moments between points of contraflexure and also reactions at interior supports. The points of contraflexure are located by placing a uniform load across the entire structure. For these moments and reactions, the results calculated from (LL#3) are compared with (LL#1) and (LL#2) results, and the critical value is selected.

[3] Used for design of interior strip only.

18.4.3.2 Pedestrian Live Load (PL)

For bridges designed for both vehicular and pedestrian live load, a pedestrian live load, PL, of 75 psf is used. However, for bridges designed exclusively for pedestrian and/or bicycle traffic, see *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges* for live load. The dynamic load allowance, IM, is not applied to pedestrian live loads **LRFD [3.6.2]**.

Pedestrian loads are not applied to an interior strip for its design. For the design of exterior strips (edge beams), any pedestrian loads that are located directly over the exterior strip width and on the cantilevered portion of the sidewalk, shall be applied to the exterior strip. See 17.2.7 for the distribution of pedestrian live loads.

18.4.4 Minimum Slab Thickness Criteria

Check adequacy of chosen slab thickness by looking at live load deflection and dead load deflection (camber) criteria, using Service I Limit State.

18.4.4.1 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits **LRFD [2.5.2.6.2]**. Live load deflections for concrete slab structures are limited to L/1200, by the Bureau of Structures. The live load deflection, Δ_{LL+IM} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.3.1 for Service I Limit State.

Place live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**. Use gross moment of inertia, I_g , based on entire slab width acting as a unit. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2. The factored resistance, R_r , is described in 18.3.4.2.2.

Then check that, $\Delta_{LL+IM} \leq R_r$ is satisfied.

18.4.4.2 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits **LRFD [5.6.3.5.2]**. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g . All dead loads are to be uniformly distributed across the width of the slab. These deflections are increased to provide for the time-dependent deformations of creep and shrinkage. Bureau of Structures currently calculates full camber as three times the

dead load deflection. Most of the excess camber is dissipated during the first year of service, which is the time period that the majority of creep and shrinkage deflection occurs. Noticeable excess deflection or structure sag can normally be attributed to falsework settlement. Use modulus of elasticity E_c = 3800 ksi, see 18.2.2 . The dead load deflection, Δ_{DL} , shall be calculated using factored loads described in 18.3.4.1 and 18.4.2. The factored resistance, R_r , is described in 18.3.4.2.3.

WisDOT exception to AASHTO:

Calculating full camber as three times the dead load deflection, as stated in paragraph above, is an exception to **LRFD [5.6.3.5.2]**. This exception, used by the Bureau of Structures, is based on field observations using this method.

Then check that, $\Delta_{DL} \leq R_r$ is satisfied.

A "Camber and Slab Thickness Diagram", "Top of Slab Elevations" table and "Survey Top of Slab Elevations" table are to be shown on the plans. See Standard 18.03 for details.

Simple-Span Concrete Slabs:

Maximum allowable camber for simple-span slabs is limited to 2 ½ inches. For simple-span slabs, Bureau of Structures practice indicates that using a minimum slab depth (ft) from the equation 1.1(S + 10) / 30, (where S is span length in feet), and meeting the live load deflection and dead load deflection (camber) limits stated in this section, provides an adequate slab section for most cases.

WisDOT exception to AASHTO:

The equation for calculating minimum slab depth for simple-spans, as stated in paragraph above, is an exception to **LRFD [Table 2.5.2.6.3-1]**. This exception, used by the Bureau of Structures, is based on past performance using this equation.

Continuous-Span Concrete Slabs:

Maximum allowable camber for continuous-span slabs is 1 ¾ inches.

18.4.5 Live Load Distribution

Live loads are distributed over an equivalent width, E, as calculated below. The equivalent distribution width applies for both live load moment and shear.

18.4.5.1 Interior Strip

Equivalent interior strip widths for slab bridges are covered in **LRFD [4.6.2.1.2, 4.6.2.3]**.

The live loads to be placed on these widths are <u>axle loads</u> (i.e., two lines of wheels) and the <u>full lane load</u>.

Single-Lane Loading: $E = 10.0 + 5.0 (L_1 W_1)^{1/2}$

Multi-Lane Loading: $E = 84.0 + 1.44(L_1 W_1)^{1/2} \leq 12.0(W)/N_L$

Where:

E	=	equivalent distribution width (in)
L_1	=	modified span length taken equal to the lesser of the actual span or 60.0 ft (ft)
W_1	=	modified edge to edge width of bridge taken to be equal to the lesser of the actual width or 60.0 ft for multi-lane loading, or 30.0 ft for single-lane loading (ft)
W	=	physical edge to edge width of bridge (ft)
N_L	=	number of design lanes as specified in **LRFD [3.6.1.1.1]**

18.4.5.1.1 Strength and Service Limit State

Use the smaller equivalent width (single-lane or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E}$$

Where:

E	=	equivalent distribution width (ft)

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore aren't used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.5.1.2 Fatigue Limit State

Use equivalent widths from single-lane loading to check fatigue stress range criteria. For the Fatigue Limit State only one design truck (Fatigue Truck) is present **LRFD [3.6.1.4]**. Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor, m, which are present in the equation for equivalent width, E, **LRFD [3.6.1.1.2]**.

The distribution factor, DF, is computed for a design slab width equal to one foot.

$$DF = \frac{1}{E(1.20)}$$

Where:

E   =   equivalent distribution width (ft)

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.5.2 Exterior Strip

Equivalent exterior strip widths for slab bridges are covered in **LRFD [4.6.2.1.4]**.

<u>For Exterior Strips without Raised Sidewalks</u>:

The exterior strip width, E, is assumed to carry one wheel line and a tributary portion of design lane load (located directly over the strip width) as shown in Figures 17.2-7 and 17.2-9.

E equals the distance between the edge of the slab and the inside face of the barrier, plus 12 inches, plus ¼ of the full strip width specified in **LRFD [4.6.2.3]**.

The exterior strip width, E, shall not exceed either ½ the full strip width or 72 inches.

Use the smaller equivalent width (single-lane or multi-lane), for full strip width, when (HL-93) live load is to be distributed, for Strength I Limit State and Service I Limit State.

The multiple presence factor, m, has been included in the equations for full strip width and therefore aren't used to adjust the distribution factor **LRFD [3.6.1.1.2]**.

<u>For Exterior Strips with Raised Sidewalks</u>:

The exterior strip width, E, is to carry a tributary portion of design lane load (when its located directly over the strip width) as in Live Load Case 1 or one wheel line as in Live Load Case 2, as shown in Figure 17.2-11.

The exterior strip width, E, shall be 72 inches.

18.4.5.2.1 Strength and Service Limit State

The distribution factor, DF, is computed for a design slab width equal to one foot.

Compute the distribution factor associated with one truck wheel line, to be applied to <u>axle loads</u>:

$$DF = \frac{(1 \text{ wheel line})}{(2 \text{ wheel lines}/\text{lane})(E)}$$

Where:

E   =   equivalent distribution width (ft)

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to <u>full lane load</u>: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\dfrac{(SWL)}{(10 \text{ ft lane load width})}\right]}{(E)}$$

Where:

E   =   equivalent distribution width (ft)

SWL   =   <u>S</u>lab <u>W</u>idth <u>L</u>oaded (with lane load) (ft) $\geq$ 0.

E – (distance from edge of slab to inside face of <u>barrier</u>)   or

E – (distance from edge of slab to inside face of <u>raised sidewalk</u>)

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.6 Longitudinal Slab Reinforcement

The concrete cover on the top bars is 2 ½ inches, which includes a ½ inch wearing surface. The bottom bar cover is 1 ½ inches. Minimum clear spacing between adjacent longitudinal bars is 3 ½ inches. The maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the slab or 18.0 inches **LRFD [5.10.3.2]**. When bundled bars are used, see **LRFD [5.10.3.1.5, 5.10.8.2.3, 5.10.8.4.2a]**.

18.4.6.1 Design for Strength

Strength Limit State considerations and assumptions are detailed in **LRFD [5.5.4, 5.6.2]**.

The area of longitudinal slab reinforcement, A_s , should be designed for strength at maximum moment locations along the structure, and for haunched slab structures, checked for strength at the haunch/slab intercepts. The area should also be checked for strength at bar reinforcement cutoff locations. This reinforcement should be designed for interior and exterior strips (edge beams) in both positive and negative moment regions. The reinforcement in the exterior strip is always equal to or greater than that required for the slab in an interior strip. Compare the reinforcement to be used for each exterior strip and select the strip with the

largest amount of reinforcement (in^2/ft). Use this reinforcement pattern for both exterior strips to keep the bar layout symmetrical. Concrete parapets, curbs, sidewalks and other appurtenances are not to be considered to provide strength to the edge beam **LRFD [9.5.1]**. The total factored moment, M_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Then calculate the coefficient of resistance, R_u :

$$R_u = M_u \,/\, \phi \; b \; d_s{}^2$$

Where:

ϕ =   0.90   (see 18.3.3.2)

b   =   12 in (for a 1 foot design slab width)

d_s =   slab depth (excl. ½ inch wearing surface) – bar clearance – ½ bar diameter (in)

Calculate the reinforcement ratio, ρ, using (R_u vs. ρ) Table 18.4-3 .

Then calculate required area,

$$A_s = \rho \; (b) \; (d_s)$$

Area of bar reinforcement per foot of slab width can be found in Table 18.4-4 .

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_u \leq M_r$ is satisfied.

The area of longitudinal reinforcement, A_s , should also be checked for moment capacity (factored resistance) along the structure, to make sure it can handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1. See Chapter 45 for details on checking the capacity of the structure for this Permit Vehicle.

18.4.6.2 Check for Fatigue

Fatigue Limit State considerations and assumptions are detailed in **LRFD [5.5.3, 5.6.1, 9.5.3]**

The area of longitudinal slab reinforcement, A_s , should be checked for fatigue stress range at locations where maximum stress range occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for fatigue stress range at bar reinforcement cutoff locations using Fatigue I Limit State. Check the reinforcement in an interior strip, where the largest number of fatigue cycles will occur.

Fatigue life of reinforcement is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

In regions where stress reversal takes place, continuous concrete slabs will be doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered.

In regions of compressive stress due to unfactored permanent loads, fatigue shall be considered only if this compressive stress is less than 1.75 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.75 times the fatigue load is tensile and exceeds $0.095 (f'_c)^{1/2}$.

The factored stress range, Q, shall be calculated using factored loads described in 18.3.5.1. The factored resistance, R_r , shall be calculated as in 18.3.5.2.1.

Then check that, Q (factored stress range) $\leq R_r$ is satisfied.

Reference is made to the design example in 18.5 of this chapter for computations relating to reinforcement remaining in tension throughout the fatigue cycle, or going through tensile and compressive stresses during the fatigue cycle.

18.4.6.3 Check for Crack Control

Service Limit State considerations and assumptions are detailed in **LRFD [5.5.2, 5.6.1, 5.6.7]**.

The area of longitudinal slab reinforcement, A_s , should be checked for crack control at locations where maximum tensile stress occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for crack control at bar reinforcement cutoff locations using Service I Limit State. Check the reinforcement in an interior and exterior strip (edge beam).

The use of high-strength steels and the acceptance of design methods where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over the bars, and spacing of reinforcement. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r , specified in **LRFD [5.4.2.6]**, for Service I Limit State. The spacing of reinforcement, s, in the layer closest to the tension face shall satisfy:

$$s \leq (700 \, \gamma_e / \beta_s \, f_{ss}) \; - \; 2 \, (d_c) \quad (in)$$

LRFD [5.6.7]

in which:

$$\beta_s = 1 + (d_c) / 0.7 \, (h - d_c)$$

Where:

γ_e =   1.00 for Class 1 exposure condition (bottom reinforcement)

γ_e =   0.75 for Class 2 exposure condition (top reinforcement)

d_c =   thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto, (in). For top reinforcement, d_c , should not include the ½" wearing surface

f_{ss} =   tensile stress in steel reinforcement (ksi) $\leq 0.6f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate (f_{ss})

h   =   overall depth of the section (in)

18.4.6.4 Minimum Reinforcement Check

The area of longitudinal slab reinforcement, A_s , should be checked for minimum reinforcement requirement at locations along the structure **LRFD [5.6.3.3]**.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , or moment capacity, at least equal to the lesser of:

M_{cr} (or) 1.33 M_u

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

f_r = $0.24 \lambda (f'c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]**

γ_1 =   1.6   flexural cracking variability factor

γ_3 =   0.67   ratio of minimum yield strength to ultimate tensile strength; for <u>A615 Grade 60 reinforcement</u>

I_g =   gross moment of Inertia (in^4)

c   =   effective slab thickness/2 (in)

M_u =   total factored moment, calculated using factored loads described in 18.3.3.1 for Strength I Limit State

λ =   concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

Select lowest value of [M_{cr} (or) 1.33 M_u] = M_L

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_L \leq M_r$ is satisfied.

18.4.6.5 Bar Cutoffs

One-half of the bar steel reinforcement required for maximum moment can be cut off at a point, where the remaining one-half has the moment capacity, or factored resistance, M_r , equal to the total factored moment, M_u , at that point. This is called the theoretical cutoff point.

Select tentative cutoff point at theoretical cutoff point or at a distance equal to the development length from the point of maximum moment, whichever is greater. The reinforcement is extended beyond this tentative point for a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. This cutoff point is acceptable, if it satisfies fatigue and crack control criteria. The continuing bars must be fully developed at this point **LRFD [5.10.8.1.2a]**.

18.4.6.5.1 Positive Moment Reinforcement

At least one-third of the maximum positive moment reinforcement in simple-spans and one-fourth of the maximum positive moment reinforcement in continuous-spans is extended along the same face of the slab beyond the centerline of the support **LRFD [5.10.8.1.2b]**.

18.4.6.5.2 Negative Moment Reinforcement

For negative moment reinforcement, the second tentative cutoff point is at the point of inflection. At least one-third of the maximum negative moment reinforcement must extend beyond this point for a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater **LRFD [5.10.8.1.2c]**.

18.4.7 Transverse Slab Reinforcement

18.4.7.1 Distribution Reinforcement

Distribution reinforcement is placed transversely in the bottom of the slab, to provide for lateral distribution of concentrated loads **LRFD [5.12.2.1]**. The criteria for main reinforcement parallel to traffic is applied. The amount of distribution reinforcement is to be determined as a percentage of the main reinforcing steel required for positive moment as given by the following formula:

$$\text{Percentage} = \frac{100\%}{\sqrt{L}} \leq 50\% \text{ maximum}$$

Where:

   L   =   span length (ft)

The above formula is conservative when applied to slab structures. This specification was primarily drafted for the relatively thin slabs on stringers.

18.4.7.2 Reinforcement in Slab over Piers

If the concrete superstructure rests on a pier cap (with columns) or directly on columns, design of transverse slab reinforcement over the pier is required. A portion of the slab over the pier is designed as a continuous transverse slab member (beam) along the centerline of the substructure. The depth of the assumed section is equal to the depth of the slab or haunch when the superstructure rests directly on columns. When the superstructure rests on a pier cap and the transverse slab member and pier cap act as a unit, the section depth will include the slab or haunch depth plus the cap depth. For a concrete slab, the width of the transverse slab member is equal to one-half the center to center spacing between columns (or 8 foot maximum) for the positive moment zone. The width equals the diameter of the column plus 6 inches for negative moment zone when no pier cap is present. The width equals the cap width for negative moment zone when a pier cap is present. Reference is made to the design example in 18.5 of this chapter for computations relating to transverse reinforcement in slab over the piers.

18.4.8 Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete.

The area, A_s , of reinforcement per foot for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.6]**

$$A_s \geq 1.30 \ (b) \ (h) \ / \ 2 \ (b+h) \ (f_y) \quad \text{and} \quad 0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	area of reinforcement in each direction and on each face (in^2/ft)
b	=	least width of component section (in)
h	=	least thickness of component section (in)
f_y	=	specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Shrinkage and temperature reinforcement shall not be spaced farther apart than 3.0 times the component thickness or 18 inches. For components greater than 36 inches thick, the spacing shall not exceed 12 inches.

All longitudinal reinforcement and transverse reinforcement in the slab must exceed required A_s (on each face and in each direction), and not exceed maximum spacing.

18.4.9 Shear Check of Slab

Slab bridges designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.12.2.1]**.

18.4.10 Longitudinal Reinforcement Tension Check

The tensile capacity check of longitudinal reinforcement on the flexural tension side of a member is detailed in **LRFD [5.7.3.5]**.

The area of longitudinal reinforcement (in bottom of slab), A_s , should be checked for tensile capacity at the abutments, for dead load and (HL-93) live load on interior and exterior strips. The reinforcement at these locations shall have the capacity to resist the tension in the reinforcement produced by shear.

The factored shear, V_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. The factored tension force, T_{fact} , from shear, to be resisted is from **LRFD [Eq'n. 5.7.3.5-2]**, where $V_s = V_p = 0$, is:

$$T_{fact} = [\, V_u \, / \, \phi_v \,] \cot \theta$$

Assume a diagonal crack would start at the inside edge of the bearing area. Assume the crack angle, θ, is 35 degrees. Calculate the distance from the bottom of slab to center of tensile reinforcement. Determine the distance D_{crack} from the end of the slab to the point at which the diagonal crack will intersect the bottom longitudinal reinforcement. Find the development length, ℓ_d , from Table 9.9-2, Chapter 9.

The nominal tensile resistance, T_{nom} , of the longitudinal bars at the crack location is:

$$T_{nom} = A_s f_y [\, D_{crack} - (\text{end cover})] \, / \, \ell_d \quad \le \quad A_s f_y$$

Then check that, $T_{fact} \le T_{nom}$ is satisfied.

If the values for T_{fact} and T_{nom} are close, the procedure for determining the crack angle, θ, as outlined in **LRFD [5.7.3.4.2]** should be used.

18.4.11 Uplift Check

Check for uplift at the abutments for (HL-93) live loads **LRFD [C3.4.1, 5.5.4.3]**. Compare the factored dead load reaction to the factored live load reaction. The reactions shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Place (HL-93) live loads in each design lane **LRFD [3.6.1.1.1]** and apply a multiple presence factor **LRFD [3.6.1.1.2]**.

18.4.12 Deflection Joints and Construction Joints

The designer should locate deflection joints in sidewalks and parapets on concrete slab structures according to the Standard *Vertical Face Parapet 'A'* in Chapter 30.

Refer to Standards *Continuous Haunched Slab* and *Continuous Flat Slab* in Chapter 18, for recommended construction joint guidelines.

18.4.13 Reinforcement Tables

Table 18.4-3 applies to: Rectangular Sections with Tension Reinforcement only

- Reinforcement Yield Strength (f_y) = 60,000 psi

- Concrete Compressive Strength (f'_c) = 4,000 psi

R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ	R_u	ρ
117.9	0.0020	335.6	0.0059	537.1	0.0098	722.6	0.0137	892.0	0.0176
123.7	0.0021	340.9	0.0060	542.1	0.0099	727.2	0.0138	896.1	0.0177
129.4	0.0022	346.3	0.0061	547.1	0.0100	731.7	0.0139	900.2	0.0178
135.2	0.0023	351.6	0.0062	552.0	0.0101	736.2	0.0140	904.4	0.0179
141.0	0.0024	357.0	0.0063	556.9	0.0102	740.7	0.0141	908.5	0.0180
146.7	0.0025	362.3	0.0064	561.8	0.0103	745.2	0.0142	912.5	0.0181
152.4	0.0026	367.6	0.0065	566.7	0.0104	749.7	0.0143	916.6	0.0182
158.1	0.0027	372.9	0.0066	571.6	0.0105	754.2	0.0144	920.7	0.0183
163.8	0.0028	378.2	0.0067	576.5	0.0106	758.7	0.0145	924.8	0.0184
169.5	0.0029	383.5	0.0068	581.4	0.0107	763.1	0.0146	928.8	0.0185
175.2	0.0030	388.8	0.0069	586.2	0.0108	767.6	0.0147	932.8	0.0186
180.9	0.0031	394.1	0.0070	591.1	0.0109	772.0	0.0148	936.9	0.0187
186.6	0.0032	399.3	0.0071	595.9	0.0110	776.5	0.0149	940.9	0.0188
192.2	0.0033	404.6	0.0072	600.8	0.0111	780.9	0.0150	944.9	0.0189
197.9	0.0034	409.8	0.0073	605.6	0.0112	785.3	0.0151	948.9	0.0190
203.5	0.0035	415.0	0.0074	610.4	0.0113	789.7	0.0152	952.9	0.0191
209.1	0.0036	420.2	0.0075	615.2	0.0114	794.1	0.0153	956.8	0.0192
214.8	0.0037	425.4	0.0076	620.0	0.0115	798.4	0.0154	960.8	0.0193
220.4	0.0038	430.6	0.0077	624.8	0.0116	802.8	0.0155	964.7	0.0194
225.9	0.0039	435.8	0.0078	629.5	0.0117	807.2	0.0156	968.7	0.0195
231.5	0.0040	441.0	0.0079	634.3	0.0118	811.5	0.0157	972.6	0.0196
237.1	0.0041	446.1	0.0080	639.0	0.0119	815.8	0.0158	976.5	0.0197
242.7	0.0042	451.3	0.0081	643.8	0.0120	820.1	0.0159	980.4	0.0198
248.2	0.0043	456.4	0.0082	648.5	0.0121	824.5	0.0160	984.3	0.0199
253.7	0.0044	461.5	0.0083	653.2	0.0122	828.8	0.0161	988.2	0.0200
259.3	0.0045	466.6	0.0084	657.9	0.0123	833.1	0.0162	992.1	0.0201
264.8	0.0046	471.7	0.0085	662.6	0.0124	837.3	0.0163	996.0	0.0202
270.3	0.0047	476.8	0.0086	667.3	0.0125	841.6	0.0164	999.8	0.0203
275.8	0.0048	481.9	0.0087	671.9	0.0126	845.9	0.0165	1003.7	0.0204
281.3	0.0049	487.0	0.0088	676.6	0.0127	850.1	0.0166	1007.5	0.0205
286.8	0.0050	492.1	0.0089	681.3	0.0128	854.3	0.0167	1011.3	0.0206
292.2	0.0051	497.1	0.0090	685.9	0.0129	858.6	0.0168	1015.1	0.0207
297.7	0.0052	502.2	0.0091	690.5	0.0130	862.8	0.0169	1018.9	0.0208
303.1	0.0053	507.2	0.0092	695.1	0.0131	867.0	0.0170	1022.7	0.0209
308.6	0.0054	512.2	0.0093	699.7	0.0132	871.2	0.0171	1026.5	0.0210
314.0	0.0055	517.2	0.0094	704.3	0.0133	875.4	0.0172	1030.3	0.0211
319.4	0.0056	522.2	0.0095	708.9	0.0134	879.5	0.0173	1034.0	0.0212
324.8	0.0057	527.2	0.0096	713.5	0.0135	883.7	0.0174	1037.8	0.0213
330.2	0.0058	532.2	0.0097	718.1	0.0136	887.9	0.0175	----	----

Table 18.4-3
R_u (psi) vs. ρ

R_u = coefficient of resistance (psi) = $M_u / \phi\, b\, d_s^2$ ρ = reinforcement ratio = $A_s / b\, d_s$

Table 18.4-4 can be used to select bar size and bar spacing to provide an adequate area of reinforcement to meet design requirements.

Bar Size Number	Nominal Dia. Inches	4 1/2"	5"	5 1/2"	6"	6 1/2"	7"	7 1/2"	8"	8 1/2"	9"	10"	12"
4	0.500	0.52	0.47	0.43	0.39	0.36	0.34	0.31	0.29	0.28	0.26	0.24	0.20
5	0.625	0.82	0.74	0.67	0.61	0.57	0.53	0.49	0.46	0.43	0.41	0.37	0.31
6	0.750	1.18	1.06	0.96	0.88	0.82	0.76	0.71	0.66	0.62	0.59	0.53	0.44
7	0.875	1.60	1.44	1.31	1.20	1.11	1.03	0.96	0.90	0.85	0.80	0.72	0.60
8	1.000	2.09	1.88	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	0.94	0.79
9	1.128	--	2.40	2.18	2.00	1.85	1.71	1.60	1.50	1.41	1.33	1.20	1.00
10	1.270	--	3.04	2.76	2.53	2.34	2.17	2.02	1.90	1.79	1.69	1.52	1.27
11	1.410	--	3.75	3.41	3.12	2.88	2.68	2.50	2.34	2.21	2.08	1.87	1.56

Table 18.4-4
Area of Bar Reinf. (in^2 / ft) vs. Spacing of Bars (in)

18.5 Standard Concrete Slab Design Procedure

18.5.1 Local Bridge Improvement Assistance Program

The Local Bridge Program was established to rehabilitate and replace, on a cost-shared basis, the most seriously deteriorating local bridges on Wisconsin's local highway and road systems. Counties, cities, villages, and towns are eligible for bridge replacement funding in accordance with the requirements in Administrative Code Trans 213. As a part of the Local Bridge Replacement Program, BOS has developed a Standard Bridge Design Tool (SBDT) to efficiently design and draft single span concrete slab bridges.

More information on the Local Bridge Improvement Assistance Program can be found at the following link:

https://wisconsindot.gov/Pages/doing-bus/local-gov/astnce-pgms/highway/localbridge.aspx.

18.5.2 Selection of Applicable Projects

On a biennial basis, locals sponsors submit applications for prospective bridge replacement projects to the WisDOT Regional Offices. The BOS Design Section assists the Regional Local Program Managers with the reviews of the applications for the appropriateness of the requested estimated bridge replacement costs. At that time, the BOS Design Section will identify candidate bridges to utilize the SBDT to streamline the bridge replacement design process. Identification of candidate bridges is based on the existing structure size, configuration, inspection and maintenance history, and known stream characteristics and flood history.

Once projects are approved for funding, the WisDOT Local Program Managers reach out to local sponsors soliciting knowledge that would preclude the use of the SBDT on those individual projects that have been identified by the BOS Design Section as candidates. If sufficient information is presented, identifying issues that will preclude the use of the tool for an identified, candidate project; then the BOS Design Section will support the conventional bridge replacement design process. However, if sufficient information is not presented, then it is the expectation that the identified candidate projects will move forward into preliminary design with the assumption that the SBDT will be utilized.

18.5.3 Use Within Other Programs

While the main focus of the SBDT is on local program usage, there may also be projects on the state system that may benefit from its use. The BOS Design Section will look for opportunities within the structures certification process to identify candidate projects on the state system to utilize the SBDT.

18.5.4 Standard Bridge Design Tool

18.5.4.1 Requirements of Designer

While the SBDT will significantly increase the efficiency with which single span slab bridge designs and plans are completed, the consultant and in-house structure designers will continue to fulfill the critical function of preliminary structure design and layout. It is expected that a structure type alternatives analysis will continue to be completed in order to verify that a single span slab bridge is the most cost-effective structure type for each project location, and that the single span slab bridge meets all site design criteria and constraints. In the event that a box culvert can be utilized, significant consideration should be given to utilizing this structure type as it is generally a more economical structure type both from an initial cost and long-term maintenance standpoint. While there would be an increase in the design fees associated with not utilizing the SBDT to make this change, those would be far outweighed over the life of the structure.

Once the structure type is verified, the preliminary type, size, and location design; hydrology and hydraulic designs; and foundation support selection remain the responsibility of the consultant. When the preliminary design and analyses are complete, the SBDT can be used to assemble the preliminary plans for submittal to the BOS Consultant Review Unit for preliminary review following the guidelines included in 6.2 and 6.5. There are no changes to the preliminary structure e-submittal contents for projects utilizing the SBDT when compared to conventional projects.

After preliminary review comments are addressed, the full set of final bridge plans can be submitted to the BOS Consultant Review unit following the guidelines included in 6.3 and 6.5. Note that design computations are not required to be submitted to BOS with the final plans unless there is a unique design feature that is added to the bridge, separate from what is automatically compiled by the SBDT. For the final quantities submittal, only those quantities not automatically compiled by the SBDT need to be submitted for review. Additionally, for the special provisions submittal, only those that need to be added in unique cases need to be submitted. For example, if a wildlife corridor is requested within the riprap slope of a standard bridge plan, then that SPV should be included in the plans and submitted for review.

The following is a list of items that need to be submitted as a part of the final e-submittal to BOS for review:

- Final Structure Plans

- QA/QC Verification Sheet

- Inventory Data Sheet

- Quantity Computations (only those not assembled by the SBDT)

- Special Provisions (only those to be added to the SBDT generated bid items)

The following is a list of items that do not need to be submitted as a part of the final e-submittal to BOS for review:

- Design Computations (unless there is a unique design feature)

- Bridge Load Rating Summary Form

- LRFD Input File

18.5.4.2 Location of Tool

The SBDT is a web-based application that can be found at the following location:

https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/design-policy-memos.aspx.

18.5.4.3 How to Utilize the Tool

The step-by-step user guide can be found at the following location:

https://wisconsindot.gov/Pages/doing-bus/local-gov/lpm/lp-standarized-bridge-plan-pilot.aspx.

18.6 Design Example

E18-1   Continuous 3-Span Haunched Slab, LRFD

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E18-1 Continuous 3-Span Haunched Slab - LRFD

A continuous 3-span haunched slab structure is used for the design example. The same basic procedure is applicable to continuous flat slabs. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. <u>Design using a slab width equal to one foot</u>. (*Example is current through LRFD Ninth Edition - 2020*)

E18-1.1 Structure Preliminary Data
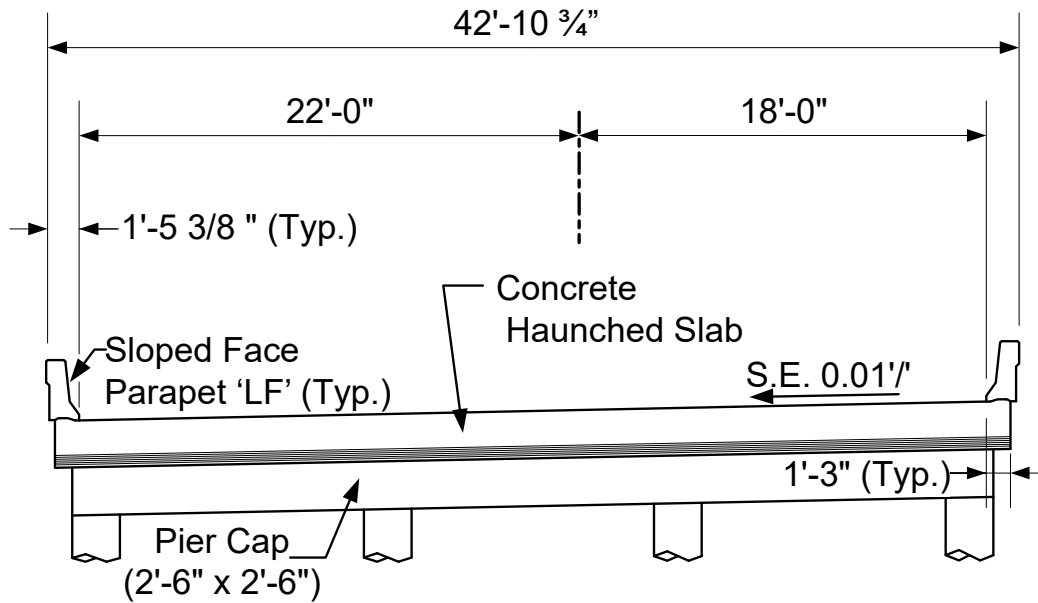
Figure E18.1

Section Perpendicular to Centerline

Live Load: HL-93
(A1) Fixed Abutments at both ends
Parapets placed after falsework is released

Geometry:

$L_1 := 38.0$ ft   Span 1

$L_2 := 51.0$ ft   Span 2

$L_3 := 38.0$ ft   Span 3

$slab_{width} := 42.5$ ft   out to out width of slab

$skew := 6$ deg   skew angle (RHF)

$w_{roadway} := 40.0$ ft   clear roadway width

Material Properties: (See 18.2.2)

$f'_c := 4$ ksi   concrete compressive strength

$f_y := 60$ ksi   yield strength of reinforcement

$E_c := 3800$ ksi   modulus of elasticity of concrete

$E_s := 29000$ ksi   modulus of elasticity of reinforcement

$n := 8$ E_s / E_c (modular ratio)

Weights:

$w_c := 150$ pcf   concrete unit weight

$w_{LF} := 387$ plf   weight of Type LF parapet (each)

E18-1.2 LRFD Requirements

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: (See 18.3.2.1)

$Q = \Sigma\eta_i \cdot \gamma_i \cdot Q_i \leq \phi \cdot R_n = R_r$ (Limit States Equation)

The value of the load modifier is:

$\eta_i := 1.0$ for all Limit States (See 18.3.2.2)

The force effect, Q_i , is the moment, shear, stress range or deformation caused by <u>applied loads</u>.

The <u>applied loads</u> from **LRFD [3.3.2]** are:

DC = dead load of slab (DC_{slab}), ½ inch wearing surface ($DC_{1/2"WS}$) and parapet dead load (DC_{para}) - (See E18-1.3)

DW = dead load of future wearing surface (DW_{FWS}) - (See E18-1.3)

LL+IM = vehicular live load (LL) with dynamic load allowance (IM) - (See E18-1.4)

The Influence of ADTT and skew on force effects, Q_i , are ignored for slab bridges (See 18.3.2.2).

The values for the load factors, γ_i , (for each <u>applied load</u>) and the resistance factors, ϕ , are found in Table E18.1.

The total factored force effect, Q , must not exceed the factored resistance, R_r . The nominal resistance, R_n , is the resistance of a component to the force effects.

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19.3.7 Construction Dimensional Tolerances

Refer to the *AASHTO LRFD Bridge Construction Specifications* for the required dimensional tolerances.

19.3.8 Prestressed I-Girder Sections

WisDOT BOS employs two prestress I-girder section families. One I section family follows the AASHTO standard section, while the other section family follows a wide flange bulb-tee, see Figure 19.3-7. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. For these sections, the cost of draping far exceeds savings in strands. See the Standard Details for the prestressed I-girder sections' draped and undraped strand patterns. Note, for the 28" prestressed I-girder section the 16 and 18 strand patterns require bond breakers.

28"

WisDOT Standard Girder Shape

36W"   45W"   54W"   72W"   82W"

WisDOT Wide Flange Girder Shapes
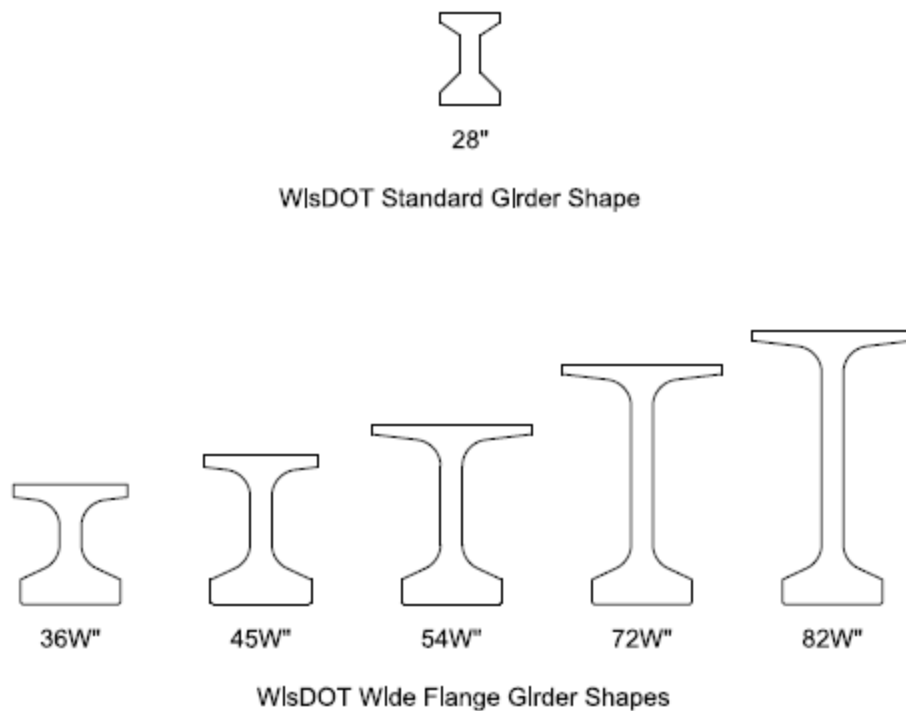
Figure 19.3-7
Prestressed I-Girder Family Details

Table 19.3-1 and Table 19.3-2 provide span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder sections.

Girder spacings are based on using low relaxation strands at $0.75f_{pu}$, concrete haunch thicknesses, slab thicknesses from Chapter 17 – Superstructures - General and a future wearing surface. For these tables, a line load of 0.300 klf is applied to the girder to account for superimposed dead loads.

Several girder shapes have been retired from standard use on new structures. These include the following sizes; 36-inch, 45-inch, 54-inch and 70-inch. These girder shapes are used for girder replacements, widening and for curved new structures where the wide flange sections are not practical. See Chapter 40 – Bridge Rehabilitation for additional information on these girder shapes.

Due to the wide flanges on the 54W, 72W and 82W and the variability of residual camber, haunch heights frequently exceed 2". An average haunch of 4" was used for all wide flange girders in the following tables. **Do not push the span limits/girder spacing during preliminary design.** See Table 19.3-2 for guidance regarding use of excessively long prestressed I-girders.

Tables are based on:

- Interior prestressed I-girders, 0.5" or 0.6" dia. strands (in accordance with the Standard Details).

- f'_c girder = 8,000 psi

- f'_c slab = 4,000 psi

- Haunch height (dead load)   = 2 ½" for 28" girder

                                                    = 4" for 36W", 45W",54W",72W" and 82W" girders

- Haunch height (section properties) = 2"

- Required f'_c girder at initial prestress < 6,800 psi

28" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	59	65
6'-6"	58	63
7'-0"	56	62
7'-6"	55	60
8'-0"	54	59
8'-6"	52	57
9'-0"	51	56
9'-6"	50	54
10'-0"	49	53
10'-6"	48	52
11'-0"	47	51
11'-6"	46	50
12'-0"	45	48

36W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	94	101
6'-6"	92	99
7'-0"	90	97
7'-6"	88	95
8'-0"	87	93
8'-6"	85	91
9'-0"	83	90
9'-6"	82	87
10'-0"	80	86
10'-6"	79	84
11'-0"	77	82
11'-6"	76	81
12'-0"	73	79

45W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	111	120
6'-6"	109	117
7'-0"	107	115
7'-6"	105	113
8'-0"	103	111
8'-6"	101	108
9'-0"	99	106
9'-6"	97	104
10'-0"	95	102
10'-6"	94	100
11'-0"	92	98
11'-6"	90	96
12'-0"	88	94

54W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	125	134
6'-6"	123	132
7'-0"	120	129
7'-6"	118	127
8'-0"	116	125
8'-6"	114	122
9'-0"	112	120
9'-6"	110	117
10'-0"	108	115
10'-6"	106	114
11'-0"	104	111
11'-6"	103	110
12'-0"	100	107

Table 19.3-1
Maximum Span Length vs. Girder Spacing

72W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	153*	164*⊗
6'-6"	150	161*⊗
7'-0"	148	158*
7'-6"	145	156*
8'-0"	143	153*
8'-6"	140	150
9'-0"	138	148
9'-6"	135	144
10'-0"	133	142
10'-6"	131	140
11'-0"	129	137
11'-6"	127	135
12'-0"	124	132

82W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	166*⊗	177*⊗
6'-6"	163*⊗	174*⊗
7'-0"	161*⊗	172*⊗
7'-6"	158*	169*⊗
8'-0"	156*	166*⊗
8'-6"	152	163*⊗
9'-0"	150	160*⊗
9'-6"	147	157*
10'-0"	145	154*
10'-6"	143	152
11'-0"	140	149
11'-6"	138	147
12'-0"	135	144

Table 19.3-2
Maximum Span Length vs. Girder Spacing

* For lateral stability during lifting these girder lengths may require pick-up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange, if required, and check the concrete strength near the lift location based on f'$_{ci}$. A note should be placed on the girder details sheet to reflect that the girder was analyzed for a potential lift at the 1/10 point.

⊗ Due to difficulty manufacturing, transporting and erecting excessively long prestressed girders, consideration should be given to utilizing an extra pier to minimize use of such girders. Approval from the Bureau of Structures is required to utilize any girder over 158 ft. long. (Currently, there is still a moratorium on the use of all 82W"). Steel girders may be considered if the number of piers can be reduced enough to offset the higher costs associated with a steel superstructure.

19.3.8.1 Prestressed I-Girder Standard Strand Patterns

The standard strand patterns presented in the Standard Details were developed to eliminate some of the trial and error involved in the strand pattern selection process. These standard strand patterns should be used whenever possible, with a straight strand arrangement preferred over a draped strand arrangement. The designer is responsible for ensuring that the selected strand pattern meets all LRFD requirements.

Section 19.3.3 discusses the key parts of the design procedure, and how to effectively use the standard strand patterns along with Table 19.3-1 and Table 19.3-2.

The amount of drape allowed is controlled by the girder size and the 2" clearance from center of strand to top of girder. See the appropriate Standard Girder Details for guidance on draping.

19.3.9 Prestressed Box Girders Post-Tensioned Transversely

These sections may be used for skews up to 30° with the transverse post-tensioning ducts placed along the skew. Skews over 30° are not recommended, but if absolutely required the transverse post-tensioning ducts should be placed perpendicular to the prestressed sections. Also for skews over 30° a more refined method of analysis should be used such as an equivalent plate analysis or a finite element analysis.

Details for transverse post-tensioning are shown in the Standard Details. Each post-tensioning duct contains three ½" diameter strands which produce a total post-tensioning force per duct of 86.7 kips.

Prestressed box girders are subject to high chloride ion exposure because of longitudinal cracking that sometimes occurs between the boxes or from drainage on the fascia girders when an open steel railing system is used. To reduce permeability the concrete mix is required to contain fly ash as stated in 503.2.2 of the Standard Specifications.

When these sections are in contact with water for 5-year flood events or less, the sections must be cast solid for long term durability. When these sections are in contact with water for the 100-year flood event or less, any voids in the section must be cast with a non-water-absorbing material.

Table 19.3-3 provides approximate span limitations for prestressed box girder sections. It also gives the section properties associated with these members. Criteria for developing these tables are shown below Table 19.3-3.

19.3.9.1 Available Prestressed Box Girder Sections and Maximum Span Lengths

Precasters have forms available to make six prestressed girder box sections ranging in depth from 12" to 42". Each section can be made in widths of 36" and 48", but 48" is more efficient and is the preferred width. Typical box section information is shown in the Standard Details.

Table 19.3-3 shows available section depths, section properties, and maximum span length. All sections have voids except the 12" deep section.

	Section No.	Section Depth (inches)	Section Area, A, (in²)	Moment of Inertia, I, (in⁴)	Section Modulus, (in³)		Torsional Inertia, J, (in⁴)	Max. Span (ft)
					S_{Top}	S_{Bottom}		
3'-0" Section Width	1	12	422	5,101	848	852	15,955	24
	2	17	452	14,047	1,648	1,657	23,797	40
	3	21	492	25,240	2,398	2,410	39,928	49
	4	27	565	50,141	3,706	3,722	68,925	58
	5	33	625	85,010	5,142	5,162	102,251	64
	6	42	715	158,749	7,546	7,573	158,033	77
4'-0" Section Width	1	12	566	6,829	1,136	1,140	22,600	25
	2	17	584	18,744	2,201	2,210	38,427	39
	3	21	624	33,501	3,184	3,197	65,395	49
	4	27	697	65,728	4,860	4,877	114,924	59
	5	33	757	110,299	6,674	6,696	173,031	68
	6	42	847	203,046	9,655	9,683	272,267	80

Table 19.3-3
Prestressed Box Girder Section Properties and Maximum Span Length

Table based on:

- HL93 loading and AASHTO LRFD Bridge Design Specifications

- Simple span

- f'$_c$ = 5 ksi and f'$_{ci}$ = 4.25 ksi

- 0.5" dia. or 0.6" dia., low relaxation prestressing strands at 0.75f'$_s$

- f'$_s$ = 270.0 ksi

- 6" min. concrete deck (which doesn't contribute to stiffness of section)

- Single slope parapet 42SS weight distributed evenly to all girder sections

- 30° skew used to compute diaphragm weight

- 2 ¾" of grout between sections

- Post-tensioning diaphragms located as stated in the Standard Details

- 30'-0" minimum clear bridge width (eleven 3'-0" sections, eight 4'-0" sections)

19.3.9.2 Decks and Overlays

There are three types of systems.

1. Reinforced Concrete Deck (design non-composite, detail composite)

2. Concrete Overlay, Grade E or C (non-composite)

3. Asphaltic Overlay with Waterproofing Membrane (not allowed)

19.3.9.3 Grout between Prestressed Box Girders

These sections are set 1" apart with a ±¼" tolerance. The space between sections is filled with a grout mix prior to post-tensioning the sections transversely. Post-tensioning is not allowed until the grout has cured for at least 48 hours and has attained a compressive strength of 3000 psi.

19.4 Field Adjustments of Pretensioning Force

When strands are tensioned in open or unheated areas during cold weather they are subject to loss due to change in temperature. This loss can be compensated for by noting the change in temperature of the strands between tensioning and initial set of the concrete. For purposes of uniformity the strand temperature at initial concrete set is taken as 80°F.

Minor changes in temperature have negligible effects on the prestress force, therefore only at strand temperatures of 50°F and lower are increases in the tensioning force made.

Since plan prestress forces are based on 75% of the ultimate for low relaxation strands it is necessary to utilize the AASHTO allowable of temporarily overstressing up to 80% to provide for the losses associated with fabrication.

The following example outlines these losses and shows the elongation computations which are used in conjunction with jack pressure gages to control the tensioning of the strands.

Computation for Field Adjustment of Prestress Force

Known:

22 - 1/2", 7 wire low relaxation strands, A_{ps} = 0.1531 in^2

P_{pj} = 710.2 kips (jacking force from plan)

T_1 = 40°F (air temperature at strand tensioning)

T_2 = 80°F (concrete temperature at initial set)

L = 300' = 3,600" (distance from anchorage to reference point)

L_1 = 240' = 2,880" (length of cast segment)

E_p = 29,000 ksi (of prestressing tendons, sample tested from each spool)

C = 0.0000065 (coefficient of thermal expansion for steel, per degree F.)

COMPUTE:

jacking force per strand = P_{pj} = 710.2/22 = 32.3 kips

DL_1 = PL/AE = 32.3 x 3600/(0.1531 x 29,000) = 26.1"

Initial Load of 1.5 Kips to set the strands

DL_2 = 1.5 x 3600/(0.1531 x 29000) = 1.22"

DL_3 = Slippage in Strand Anchors = 0.45" (Past Experience)

DL_4 = Movement in Anchoring Abutments = 0.25" (Past Experience)

Table of Contents

24.1 Introduction

Steel girders are recommended due to depth of section considerations for short span structures and due to their economy in comparison with other materials or structure types for longer span structures.

24.1.1 Types of Steel Girder Structures

This chapter considers the following common types of steel girder structures:

- Plate girder

- Rolled girder

- Box girder

A plate girder structure is selected over a rolled girder structure for longer spans or when greater versatility is required. Generally rolled girders are used for web depths less than 36" on short span structures of 80' or less.

24.1.2 Structural Action of Steel Girder Structures

Box girder, rolled girder and plate girder bridges are primarily flexural structures which carry their loads by bending between the supports. The degree of continuity of the steel girders over their intermediate supports determines the structural action within the steel bridge. The main types of structural action are as follows:

- Simply-supported structures

- Multiple-span continuous structures

- Multiple-span continuous hinged structures

Simply-supported structures are generally used for single, short-span structures. Multiple-span steel girder structures are designed as continuous spans. When the overall length of the continuous structure exceeds approximately 900', a transverse expansion joint is provided by employing girder hinges and a modular watertight expansion device.

The 900' guideline is based on the abutments having expansion bearings and a pier or piers near the center of the continuous segment having fixed bearings. More than one fixed pier shall be used when four or more piers are utilized or when a steep grade (greater than 3%) exists. When one abutment has fixed bearings, see Chapter 12 – Abutments for the limitation on the length of a continuous segment.

24.1.3 Fundamental Concepts of Steel I-Girders

This section describes basic concepts of I-girder sections to aid in understanding the design provisions for steel I-sections presented in *AASHTO LRFD*. This section is cursory in nature.

The behavior of non-composite steel I-section members subject to flexure is similar to the behavior of composite I-section members in negative flexure. A qualitative bending moment versus rotation relationship for a homogeneous compact web section is presented Figure 24.1-1.

A homogeneous section is defined as a section in which the flanges and web have the same nominal yield strength.

In *AASHTO LRFD*, a compact web section is defined as a non-composite section (or a composite section in negative flexure) that has a web with a slenderness at or below which the section can achieve a maximum flexural resistance, M_{max}, equal to the plastic moment, M_p, prior to web bend-buckling having a statistically significant influence on the response. In addition, specific steel grade, ductility, flange slenderness and lateral bracing requirements must also be satisfied. Compact web sections are typically shallower sections, with thicker webs, than non-compact sections. Compact web sections are often rolled beams or welded girder sections with proportions similar to rolled beams.
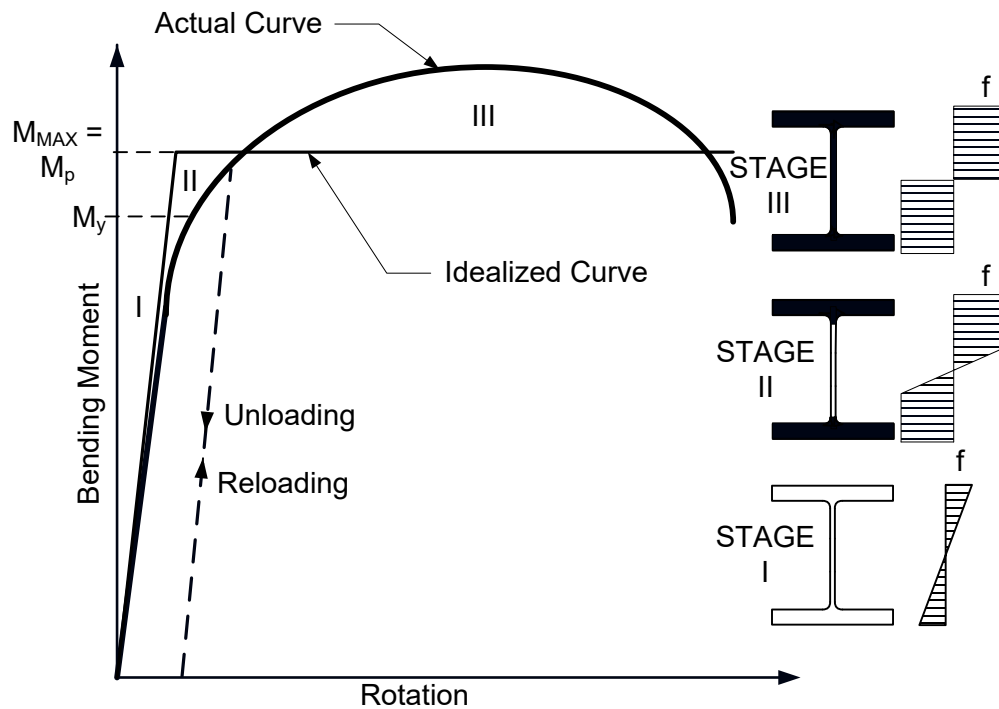
Figure 24.1-1
Bending Moment versus Rotation for Homogeneous Compact Web Section

Proceeding along the actual curve shown in Figure 24.1-1, the initial Stage I behavior represents completely elastic behavior. As the section approaches the theoretical yield moment, M_y, the presence of residual stresses will result in some inelastic behavior in the outer fibers of the cross section before the calculated M_y is reached. At Stage II, yielding continues and begins to progress throughout the section as the section approaches the plastic moment, M_p. At Stage III, the entire cross section has yielded; that is, each component of the cross

section is assumed to be at F_y. The idealized curve shown in Figure 24.1-1 is assumed for design. The dotted line shown in Figure 24.1-1 illustrates the behavior of a member that is loaded with a moment greater than M_y and then unloaded.

Figure 24.1-2 shows a moment versus rotation relationship for a homogeneous slender web section. In *AASHTO LRFD*, a slender web section is defined as a non-composite section (or a composite section in negative flexure) that has a web with a slenderness at or above which the theoretical elastic bend-buckling stress in flexure is reached in the web prior to reaching the yield strength of the compression flange. Because web bend-buckling is assumed to occur in such sections, a web load-shedding factor, R_b, must be introduced to account for the effect of the post-bend-buckling resistance or redistribution of the web compressive stresses to the compression flange resulting from the bend-buckling of the web **LRFD [6.10.1.10.2]**.

The maximum flexural resistance, M_{max}, is taken as the smaller of $R_b M_{yc}$ and M_{yt} for a homogeneous slender-web section, where M_{yc} and M_{yt} are the yield moments with respect to the compression and tension flanges, respectively. Like a compact web section, residual stresses will contribute to yielding and some inelastic behavior will occur prior to reaching M_{max}, as shown in Figure 24.1-2. However, unlike a compact web section, a slender web section has little or no available inelastic rotation capacity after reaching M_{max}. Therefore, the flexural resistance drops off quite rapidly after reaching M_{max}, and redistribution of moments is not permitted when these sections are used at interior piers.
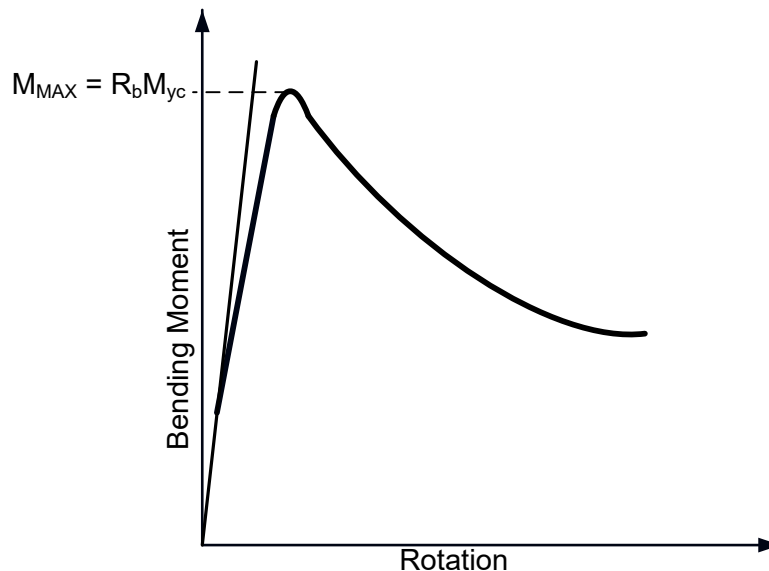
Figure 24.1-2
Moment versus Curvature for Homogeneous Slender Web Section

Sections with a web slenderness between the slenderness limits for a compact web and a slender web section are termed non-compact web sections. This represents a change from previous *AASHTO Specifications*, which defined sections as either compact or non-compact and did not distinguish between a non-compact and a slender web.

In *AASHTO LRFD*, a non-compact web section is defined as a non-composite section (or a composite section in negative flexure) that has a web satisfying steel grade requirements and with a slenderness at or below the limit at which theoretical elastic web bend-buckling does not occur for elastic stress levels, computed according to beam theory, smaller than the limit of the nominal flexural resistance.

Because web bend-buckling is not assumed to occur, R_b is taken equal to 1.0 for these sections. The maximum flexural resistance of a non-compact web section, M_{max}, is taken as the smaller of $R_{pc}M_{yc}$ and $R_{pt}M_{yt}$. It falls between M_{max} for a compact web and a slender web section as a linear function of the web slenderness ratio. R_{pc} and R_{pt} are termed web plastification factors for the compression and tension flange, respectively. The web plastification factors are essentially effective shape factors that define a smooth linear transition in the maximum flexural resistance between M_y and M_p.

The basic relationship between M_{max} and the web slenderness $2D_c/t_w$ given in *AASHTO LRFD* is presented in Figure 24.1-3. Figure 24.1-3 assumes that yielding with respect to the compression flange controls. The relationship between M_{max} and web slenderness is defined in terms of all three types of sections – compact web, non-compact web and slender web.
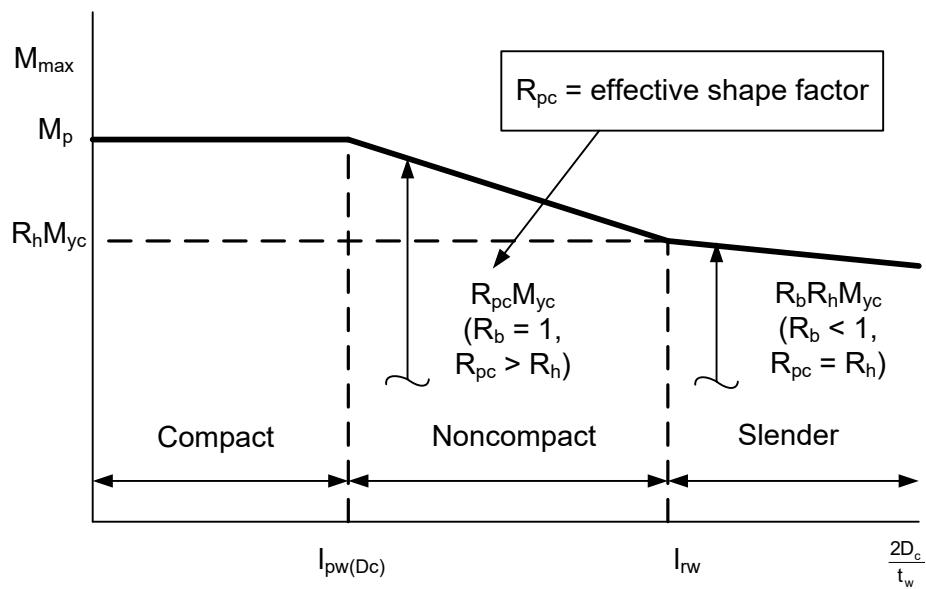
Figure 24.1-3
M_{max} versus Web Slenderness

In *AASHTO LRFD*, the flexural resistance for slender web sections is expressed in terms of stress. For compact web and non-compact web sections, in which the maximum potential flexural resistance equals or exceeds M_y, the resistance equations are more conveniently expressed in terms of bending moment.

Lateral torsional buckling can result if the compression flange of an I-section member does not have adequate lateral support. The member deflects laterally in a torsional mode before the

compressive bending stress reaches the yield stress. Lateral torsional buckling is illustrated in Figure 24.1-4.
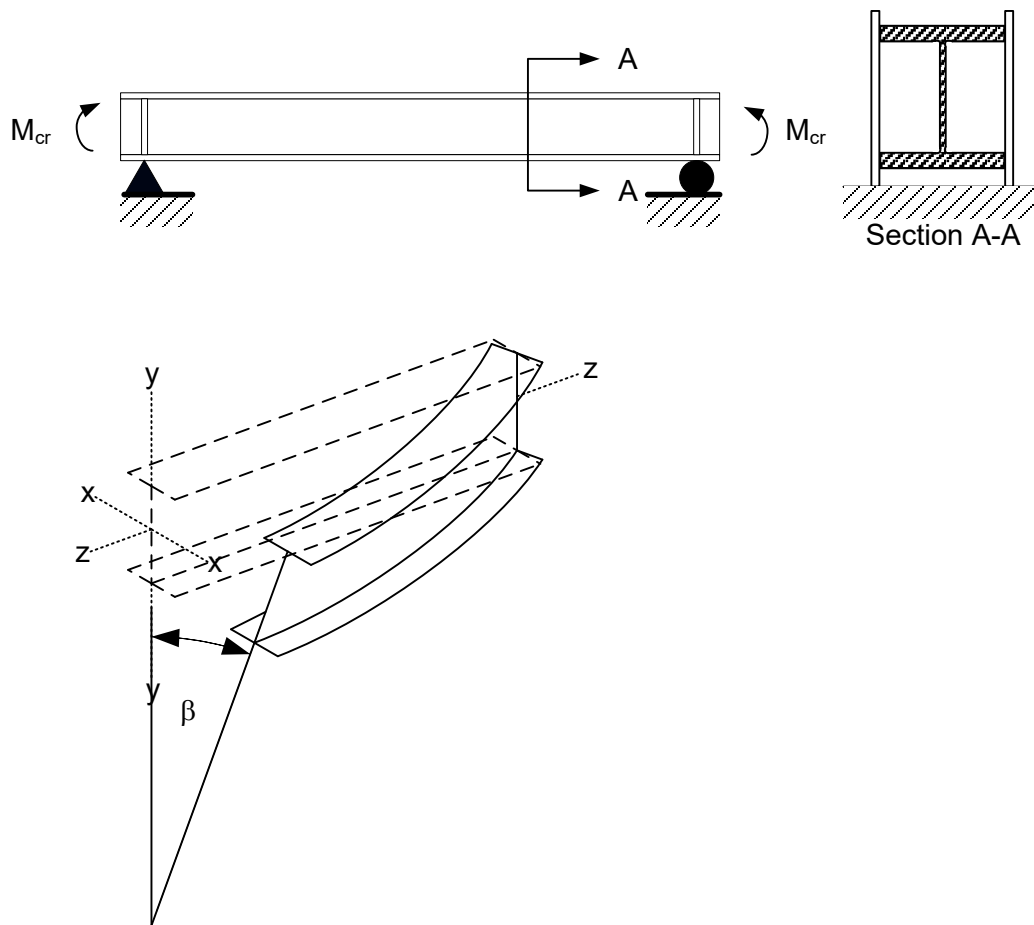
Figure 24.1-4
Lateral Torsional Buckling in a Doubly Symmetric I-section Member

As presented in Figure 24.1-5, *AASHTO LRFD* has adopted a simple linear expression to approximate the lateral-torsional buckling resistance of discretely braced compression flanges in the inelastic range. Figure 24.1-5 also shows the basic form of the flange local buckling equations in *AASHTO LRFD*, which is similar to the form of the lateral-torsional buckling equations.
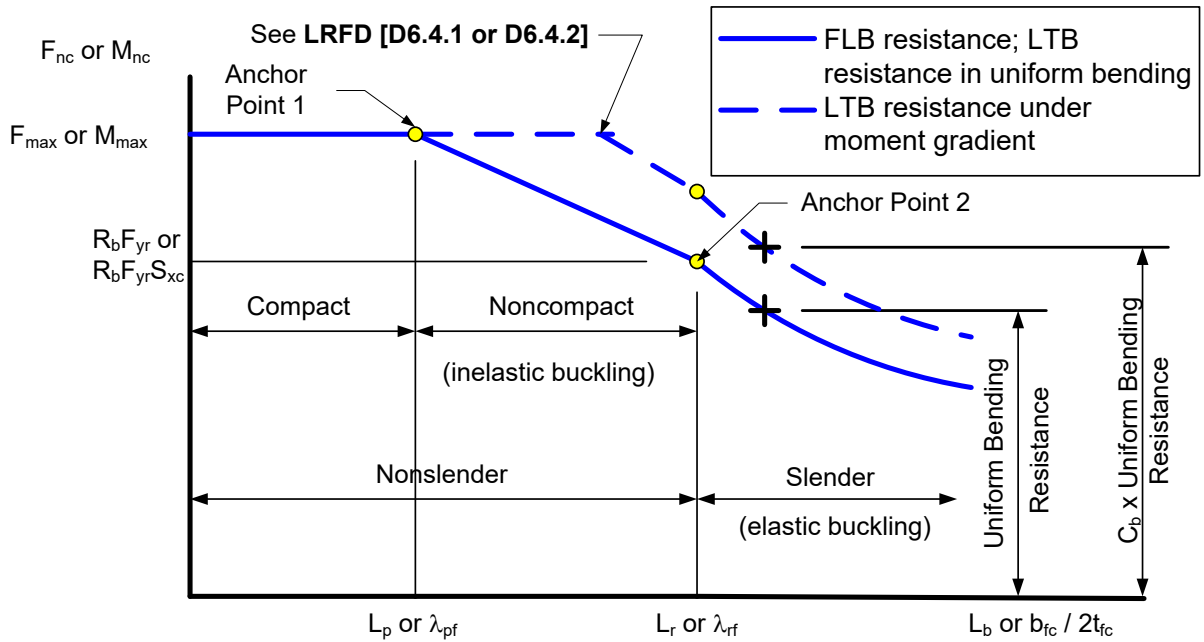
Figure 24.1-5
Form of the Compression-Flange Resistance Equations in *AASHTO LRFD*

24.2 Materials

Structural steels currently used conform to ASTM A709 Specifications designated Grades 36, 50 and 50W. *AASHTO LRFD* gives the necessary design information for each grade of steel. Steel girders may utilize High-Performance Steel (HPS); however it may come at a premium price due to the limited number of mills that are rolling HPS. The limited number of mills may also have adverse effects on the delivery schedule.

HPS is currently produced by either quenching and tempering (Q&T) or by thermo-mechanical-controlled-processing (TMCP). TMCP HPS is currently available in plate thicknesses up to 2" and in maximum plate lengths from approximately 50' to 125' depending on weights. Q&T HPS is available in plate thicknesses from 2" to 4" (or less for larger plate widths), but because of the furnaces that are used in the tempering process, it is subject to a maximum plate-length limitation of 600" (50') or less, depending on weights. Therefore, whenever Q&T HPS is used (generally when HPS plates over 2" in thickness are specified), the maximum plate-length limitation should be considered when laying out flange (and web) transitions in a girder.

For fracture toughness, HPS provides significant toughness improvements given, that by default, Charpy V-notch requirements satisfy the more stringent Zone 3 requirements in all temperature zones. For welding, most of the bridge steels specified in the ASTM A709 Specifications can be welded without special precautions or procedures. However, special procedures should be followed to improve weldability and ensure high-quality welds when HPS is used.

Hybrid girder design utilizing HPS Grade 70 steel (Grade 70 is only available in HPS) for the flanges and Grade 50 steel for the web may be considered as a viable alternative. Such an arrangement has recently proven to be a popular option, primarily in regions of negative flexure.

For unpainted structures over stream crossings, Grade 50W weathering steel is recommended throughout.

Cracks have been observed in steel girders due to fabrication, fatigue, brittle fractures and stress corrosion. To insure against structural failure, the material is tested for plane-strain fracture toughness. As a result of past experience, the Charpy V-notch test is currently required on all grades of steel used for girders.

Plate width and length availability is an important consideration when it comes to sizing girder flanges. The availability of plate material varies from mill to mill. Generally, plates are available in minimum widths ranging from 48" to 60" and in maximum widths ranging from 150" to 190". *AASHTO/NSBA Steel Bridge Collaboration*, "*Guidelines to Design for Constructibility, G12.1*" (2020) contains some example plate length and width availability information from a single mill. However, a fabricator and/or mill should be consulted regarding the most up-to-date plate availability information. The maximum available plate length is generally a function of the plate width and thickness, steel grade and production process.

For additional information about plate widths and lengths, including maximum sizes for shipping and erection, see 24.4.6.2.

For additional information about materials, see Chapter 9 – Materials.

24.2.1 Bars and Plates

Bars and plates are grouped under flat rolled steel products that are designated by size as follows:

- Bars – 8" or less in width

- Plates – over 8" in width

WisDOT policy item:

AASHTO LRFD allows a minimum thickness of 5/16" for most structural steel members. Current WisDOT policy is to employ a minimum thickness of 7/16" for primary members and a minimum of 3/8" for secondary structural steel members.

Optional splices are permitted on plates which are detailed over 60' long. Refer to the latest steel product catalogs for steel sections and rolled stock availability.

24.2.2 Rolled Sections

A wide variety of structural steel shapes are produced by steel manufacturers. Design and detail information is available in the *AISC Manual of Steel Construction*, and information on previously rolled shapes is given in *AISC Iron and Steel Beams 1873 to 1952*. Refer to the latest steel product catalogs for availability and cost, as some shapes are not readily available and their use could cause costly construction delays.

24.2.3 Threaded Fasteners

The design of bolted connections is covered in **LRFD [6.13.2]**. As specified in **LRFD [6.13.2.1]**, bolted steel parts must fit solidly together after the bolts are tightened. The bolted parts may be coated or uncoated. It must be specified in the contract documents that all joint surfaces, including surfaces adjacent to the bolt head and nut, be free of scale (except for tight mill scale), dirt or other foreign material. All material within the grip of the bolt must be steel.

High-strength bolts are installed to have a specified initial tension, which results in an initial pre-compression between the joined parts. At service load levels, the transfer of the loads between the joined parts may then occur entirely via friction, with no bearing of the bolt shank against the side of the hole. Until the friction force is overcome, the shear resistance of the bolt and the bearing resistance of the bolt hole will not affect the ability to transfer the load across the shear plane between the joined parts.

In general, high-strength bolted connections designed according to *AASHTO LRFD* will have a higher reliability than the connected parts because the resistance factors for the design of bolted connections were selected to provide a higher level of reliability than those chosen for member design. Also, the controlling strength limit state in the connected part (for example, yielding or deflection) is typically reached well before the controlling strength limit state in the
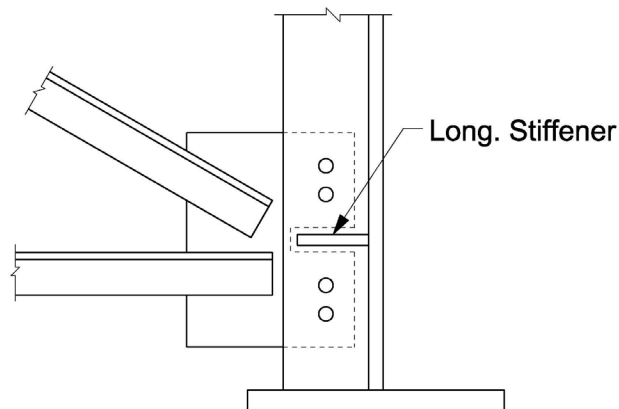
Figure 24.4-1
Cross Frame Where Longitudinal Stiffener is Used

24.4.5.2 End Diaphragms

End diaphragms are placed horizontally along the abutment end of beams or girders and at other points of discontinuity in the structure. Channel sections are generally used for end diaphragms, and they are designed as simply-supported edge beams. The live load moment plus dynamic load allowance is determined by placing one wheel load or two wheel loads 4' apart and correcting for the skew angle at the center line of the member. Generally, the dead load moment of the overlying slab and diaphragm is insignificant and as such is neglected. End diaphragm details and dimensions are given on Standard for *End Diaphragms*.

End diaphragms are either bolted or welded to gussets attached to the girders at points of discontinuity in the superstructure. The gusset plates are bolted to the bearing stiffeners. The same connection detail is used throughout the structure. The connections are designed for shear only where joined at a web since very little moment is transferred without a flange connection. The connection is designed for the shear due to live load plus dynamic load allowance from the wheel loads.

24.4.5.3 Lower Lateral Bracing

Lateral bracing requirements for the bottom flanges are to be investigated. Bureau of Structures (BOS) practice is to eliminate the need for bracing by either increasing flange sizes or reducing the distance between cross frames. The controlling case for this stress is usually at a beam cutoff point. At cutoff points, the condition of maximum stress exists with the smallest flange size, where wind loads have the greatest effect. A case worth examining is the temporary stress that exists in top flanges during construction. Top flange plates, which are often only 12" wide, can be heavily stressed by wind load. A temporary bracing system placed by the contractor may be in order.

On an adjacent span to one requiring lower lateral bracing, the bracing is extended one or two panel lengths into that span. The lower lateral bracing system is placed in the exterior bays of

the bridge and in at least 1/3 of the bays of the bridge. On longer spans, the stresses caused by wind load during construction will generally govern the member size.

Curved girders in Wisconsin generally do not have extremely long span lengths, and the curvature of the girders forms an arch which is usually capable of resisting the wind forces prior to placing the slab.

24.4.6 Girder Selection

The exterior girder section is always designed and detailed such that it is equal to or larger than the interior girder sections. Guidelines for ratios of girder depth to length of span are provided in 24.4.2. The following criteria are used to determine the selection and sizes of girder sections. For additional rules of thumb regarding economical design considerations, see 24.6.2.

24.4.6.1 Rolled Girders

Rolled girders without cover plates are preferred. Cover plates are not recommended due to fatigue considerations and higher fabrication costs.

24.4.6.2 Plate Girders

Basic cross-section proportion limits for flanges of steel I-girders are specified in **LRFD [6.10.2.2]**. The limits apply to both tension and compression flanges. The minimum width of flanges, b_f, is specified as:

$$b_f \geq D/6$$

Where:

D   =   Web depth

This limit is a lower limit, and flange widths should not be set based on this limit. Practical size flanges should easily satisfy this limitation based on satisfaction of other design criteria. Fabricators prefer that flange widths never be less than 12" to prevent distortion and cupping of the flanges during welding, which sets a practical lower limit.

Composite design has led to a significant reduction in the size of compression flanges in regions of positive flexure as economical composite girders normally have smaller top flanges than bottom flanges. In regions of positive flexure during deck placement, more than half the web is typically in compression. As a result, maximum moments generated during the deck-casting sequence, coupled with top compression flanges that are too narrow, can lead to out-of-plane distortions of the compression flanges and web during construction. The following relationship from **LRFD [C6.10.2.2]** is a suggested guideline on the minimum top compression flange width, b_{tfs}, that should be provided in these regions to help minimize potential problems in these cases:

$$b_{tfs} \geq L_{fs} / 85$$

Where:

> b_{tfs} =   Smallest top flange width within the unspliced individual girder field section under consideration (in.)
>
> L_{fs} =   Length of the unspliced individual girder field section under consideration (in.)

Satisfaction of this simple guideline can also help ensure that individual field sections will be stable for handling both in the fabrication shop and in the field. Adherence to this guideline can also facilitate erection without any required special stiffening trusses or falsework. It is recommended that the above two equations be used to establish a minimum required top-flange width in regions of positive flexure in composite girders.

As a practical matter, fabricators order flange material from wide plate, typically between 72" and 96" wide. They either weld the shop splices in the individual flanges after cutting them to width or they weld the different thickness plates together to form one wide plate and then strip the individual flanges. In the latter case, the individual flange widths must be kept constant within an individual shipping piece, which is preferred. Changing of flange widths at shop splices should be avoided if at all possible. Stripping the individual flanges from a single wide plate allows for fewer weld starts and stops and results in only one set of run-on and run-off tabs. It is estimated that up to 35% of the labor required to join the flanges can be saved by specifying changes in thickness rather than width within a field section.

A fabricator will generally order plate with additional width and length for cutting tolerance, sweep tolerance and waste. Waste is a particular concern when horizontally curved flanges are cut curved. The engineer should give some consideration as to how the material might be ordered and spliced; a fabricator can always be consulted for assistance. Flanges should be sized (including width, thickness and length) so that plates can be ordered and spliced with minimal waste. *AASHTO/NSBA Steel Bridge Collaboration*, "*Guidelines to Design for Constructability, G12.1*" (2020) is a free publication available from AASHTO which contains some specific recommendations and illustrative examples related to this issue.

The following additional guidelines are used for plate girder design and detailing:

1. Maximum change in flange plate thickness is 1" and preferably less.

2. The thinner plate is not less than 1/2 the thickness of the thicker flange plate.

3. Plate thicknesses are given in the following increments:

4. 1/16" up to 1"

5. 1/8" between 1" and 2"

6. 1/4" above 2"

7. Minimum plate size on the top flange of a composite section in the positive moment region is variable depending on the depth of web, but not less than 12" x ¾" for web depths less than or equal to 66" and 14" x ¾" for web depths greater than 66". Thinner plates become wavy and require extra labor costs to straighten within tolerances.

8. For plate girder flange widths, use 2" increments.

9. For plate girder web depths, use 3" increments.

10. Changes in plate widths or depths are to follow recommended standard transition distances and/or radii. The minimum size flange plates of 16" x 1 1/2" at the point of maximum negative moment and 16" x 1" for the bottom flange at the point of maximum positive moment are recommended for use on plate girders. The use of a minimum flange width on plate girders is necessary to maintain adequate stiffness in the girder so it can be fabricated, transported and erected. Deeper web plates with small flanges may use less steel, but they create problems during fabrication and construction. However, flange sizes on plate girders with web depths 48" or less may be smaller.

11. Flange plate sizes are detailed based on recommended maximum span lengths given in Table 24.4-1 for parallel flanged girders. The most economical girder is generally the one having the least total weight but is determined by comparing material costs and welding costs for added stiffener details. Plates over 60'-90' (depending on thickness and material) are difficult to obtain, and butt splices are detailed to limit flange plates to these lengths or less. It is better to detail more flange butt splices than required and leave the decision to utilize them up to the fabricator. All butt splices are made optional to the extent of available lengths, and payment is based on the plate sizes shown on the plans. As previously described, detail flange plates to the same width and vary the thicknesses. This allows easier fabrication when cutting plate widths. Change widths, if necessary, only at field splices.

12. Minimum web thickness is 7/16" for girder depths less than or equal to 60". An economical web thickness usually has a few transverse stiffeners. Refer to 24.10 for transverse stiffener requirements. Due to fatigue problems, use of longitudinal stiffeners for plate girders is not encouraged.

24.4.7 Welding

Welding design details shall conform to current requirements of *Bridge Welding Code: AASHTO/AWS-D1.5*. Weld details are not shown on the plans but are specified by using standard symbols as given on Figure 24.4-2 and Figure 24.4-3. Weld sizes are based on the size required due to stress or the minimum size for plate thicknesses being connected.

24.5.2 Charpy V-Notch Impact Requirements

Recognizing the need to prevent brittle fracture failures of main load-carrying structural components, AASHTO adopted provisions for Charpy V-Notch impact testing in 1974. Impact testing offers an important measure of material quality, particularly in terms of ductility. Brittleness is detected prior to placing the material in service to prevent member service failures. Wisconsin *Standard Specifications for Highway and Structure Construction* require Charpy V-Notch tests on all girder flange and web plates, flange splice plates, hanger bars, links, rolled beams and flange cover plates. Special provisions require higher Charpy V-Notch values for non-redundant structure types.

For the Charpy V-Notch impact test, small, notched steel specimens are loaded at very high strain rates as the specimen absorbs the impact from a pendulum. The maximum height the pendulum rises after impact measures the amount of energy absorbed in foot-pounds.

The AASHTO fracture control plan uses three different temperature zones (designated Zones 1, 2 and 3) to qualify the fracture toughness of bridge steels. The three zones are differentiated by their minimum operating (or service) temperatures, which are given in **LRFD [Table 6.6.2.1-2]**. In Wisconsin, use Zone 2 requirements.

Separate fracture toughness requirements are given in **LRFD [Table C6.6.2.1-1]** for non-fracture-critical and fracture-critical members (or components). A fracture-critical member (FCM) is defined as a component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function. FCMs are subject to more stringent Charpy V-Notch fracture toughness requirements than non-fracture-critical members. For FCMs, High Performance Steel (HPS) shall be used with Zone 2 requirements.

According to **LRFD [6.6.2.2]**, the engineer has the responsibility to identify all bridge members or components that are fracture critical and clearly delineate their location on the contract plans. Examples of FCMs in bridges include certain truss members in tension, suspension cables, tension components of girders in two-girder systems, pin and link systems in suspended spans, cross girders and welded tie girders in tied-arches. In addition, any attachment having a length in the direction of the tension stress greater than 4 inches and welded to the tension area of a component of a FCM is also to be considered fracture critical.

24.5.3 Non-Redundant Type Structures

Previous AASHTO fatigue and fracture toughness provisions provided satisfactory fracture control for multi-girder structures when employed with good fabrication and inspection practices. However, concern existed that some additional factor of safety against the possibility of brittle fracture should be provided in the design of non-redundant type structures such as single-box and two-box girders, two-plate girders or truss systems where failure of a single element could cause collapse of the structure. A case in point was the collapse of the Point Pleasant Bridge over the Ohio River. HPS shall be used for non-redundant structures.

Primary factors controlling the susceptibility of non-redundant structures to brittle fracture are the material toughness, flaw size and stress level. One of the most effective methods of reducing brittle fracture is lowering the stress range imposed on the member. AASHTO provides an increased safety factor for non-redundant members by requiring a shift of one

range of loading cycles for fatigue design with corresponding reduction of stress range for critical stress categories. The restrictive ranges for certain categories require the designer to investigate the use of details which do not fall in critical stress categories or induce brittle fracture. For non-fracture-critical members including bolted tie girders found in tied arch bridges, multiple box girder structures (3 boxes) and hanger plates, HPS shall also be used.

As per a FHWA directive, two-girder box girder structures are to be considered non-redundant.

For I-girder bridges, other than pedestrian or other unusual structures, four or more girders shall be used.

check the maximum rolling lengths of plates to see if additional butt splices are required. The fabricator may assume the cost of extending the heavier plate and eliminating the butt splice; this option has been used by fabricators on numerous occasions. Shim plates are provided at the bearing to allow for either option. A common rule of thumb is to use no more than three plates (two shop splices) in the top or bottom flange of field sections up to 130 feet long. In some cases, a single flange plate size can be carried through the full length of the field section. Estimate field splice locations at approximately the 7/10 point of continuous spans.

- Flange widths – Flange widths should remain constant within field sections. The use of constant flange widths simplifies construction of the deck. The unsupported length in compression of the shipping piece divided by the minimum width of the compression flange in that piece should be less than approximately 85. High bearing reactions at the piers of continuous girders may govern the width of the bottom flange.

- Flange transitions – It is good design practice to reduce the flange cross-sectional area by no more than approximately one-half of the area of the heavier flange plate. This reduces the build-up of stress at the transition.

- Haunched girders – On haunched plate girders, the length of the parabolic haunch is approximately 1/4 of the span length. The haunch depth is 1 1/2 times the midspan depth.

It should be noted that during the optimization process, minor adjustments can be made to the plate sizes and transition locations without needing to recompute the analysis results. However, if significant adjustments are made, such that the moments and shears would change significantly, then a revised analysis is required.

24.6.3 Compute Section Properties

See 17.2.11 for determining composite slab width.

For a composite superstructure, several sets of section properties must be computed. The initial dead loads (or the non-composite dead loads) are applied to the girder-only section. The superimposed dead loads are applied to the composite section based on a modular ratio of 3n, as described in **LRFD [6.10.1.1.1]**. The live loads are applied to the composite section based on a modular ratio of n.

For girders with shear connectors provided throughout their entire length and with slab reinforcement satisfying the provisions of **LRFD [6.10.1.7]**, stresses due to loads applied to the composite section for the Fatigue I and Service II limit states may be computed using the short-term composite section, based on a modular ratio of n, assuming the concrete slab to be fully effective for both positive and negative flexure.

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not.

For LRFD, Wisconsin places shear connectors in both the positive and negative moment regions of continuous steel girder bridges, and both regions are considered composite in analysis and design computations. Negative flexure concrete deck reinforcement is considered in the section property calculations.

24.6.4 Compute Dead Load Effects

The girder must be designed to resist the dead load effects, as well as the other load effects. Various types of dead loads and their corresponding load factors are described in 17.2.4 and 17.2.5.

For the steel girder, the dead load per unit length varies due to the change in plate sizes. The moments and shears due to the weight of the steel girder can be computed using analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

Distribution of dead load to the girders is described in 17.2.8.

The stiffness of the composite section is used for determining live load and composite dead load moments and shears. When computing live load values, the composite section is based on n, and when computing composite dead load values, the composite section is based on 3n. Non-composite dead load moments and shears are computed based on the stiffness of the non-composite steel section.

24.6.5 Compute Live Load Effects

The girder must also be designed to resist the live load effects. The live load consists of an HL-93 loading. Similar to the dead load, the live load moments and shears for an HL-93 loading can be obtained from an analysis computer program.

For all limit states other than fatigue and fracture, the dynamic load allowance, IM, is 0.33. However, it is important to note that the dynamic load allowance is applied only to the design truck or tandem. The dynamic load allowance is not applied to pedestrian loads or to the design lane load.

Live load distribution factors must be computed as specified in **LRFD [4.6.2.2]**, as shown in Table 24.6-1.

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For beams with variable moment of inertia, the longitudinal stiffness parameter, Kg (**LRFD [Eq'n 4.6.2.2.1-1]**), shall be based on a weighted average of properties, over the entire length of the bridge.

In addition to computing the live load distribution factors, their ranges of applicability must also be checked. If they are not satisfied, then conservative assumptions must be made based on sound engineering judgment. Additional information about distribution of live load to the girders is presented in 17.2.8.

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.
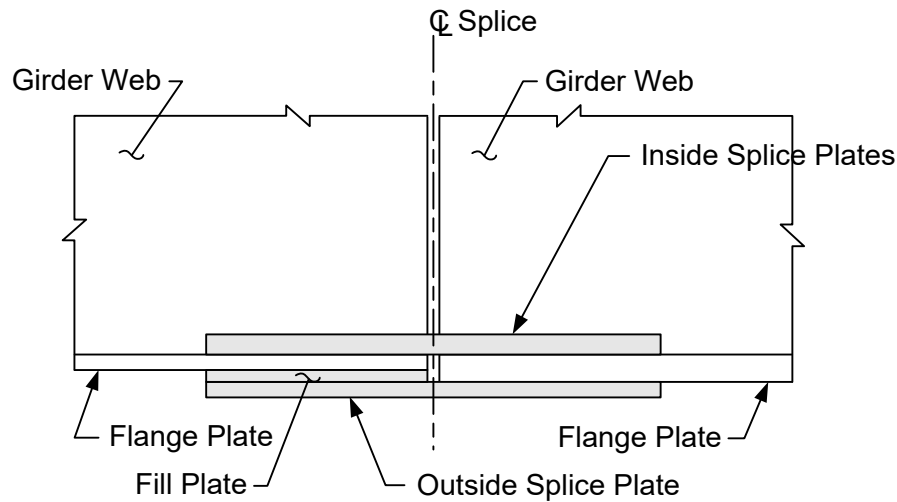
Figure 24.8-1
Bottom Flange Splice Configuration

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.
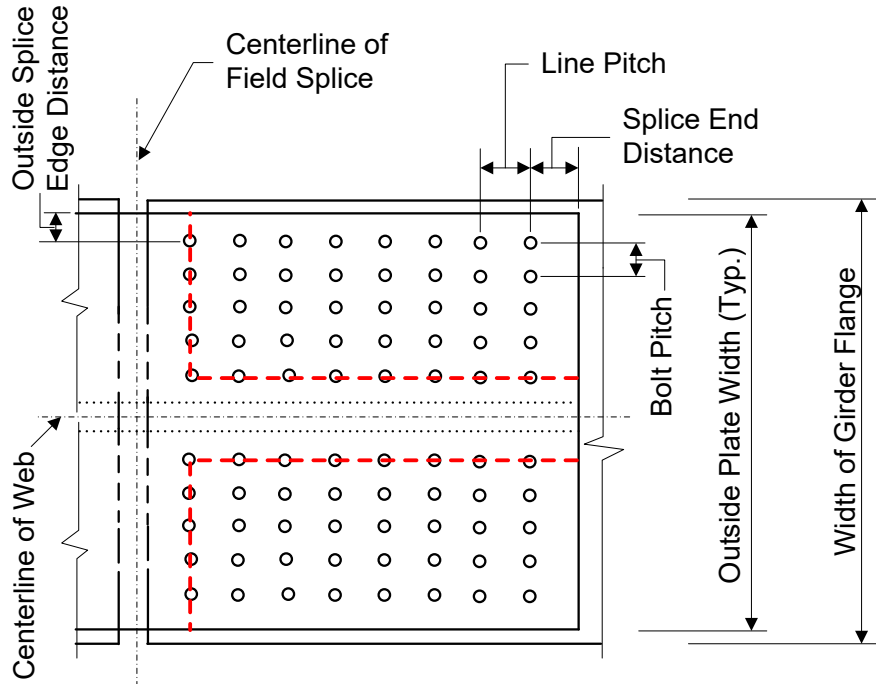
Figure 24.8-2
Double – L Block Shear Path, Flange and Splice Plates
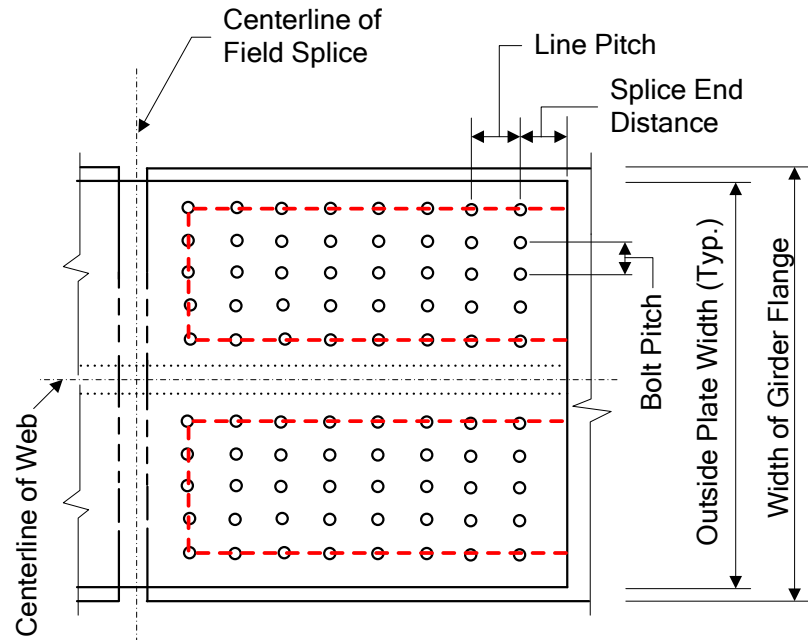
Figure 24.8-3
Double – U Block Shear Path, Flange and Splice Plates

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.
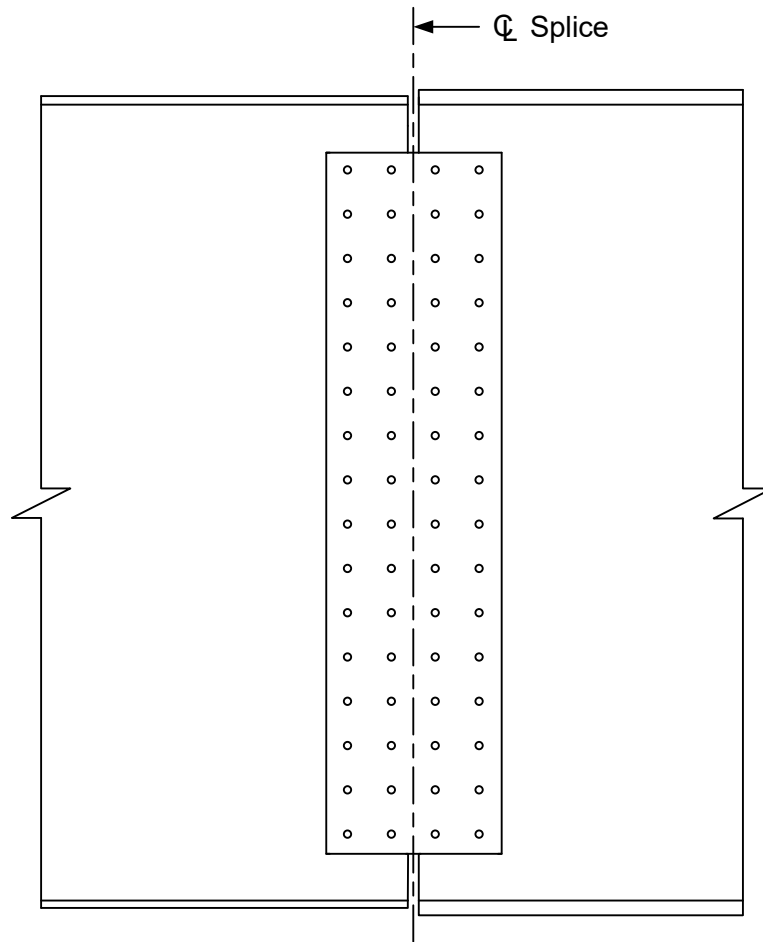
Figure 24.8-4
Web Splice Configuration

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.
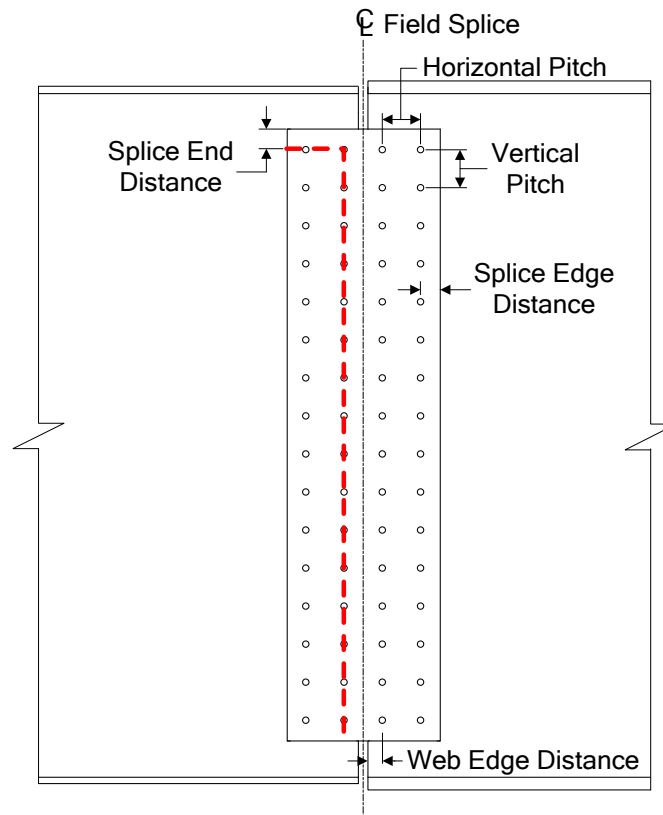
Figure 24.8-5
Block Shear Path, Web Splice

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

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- 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.
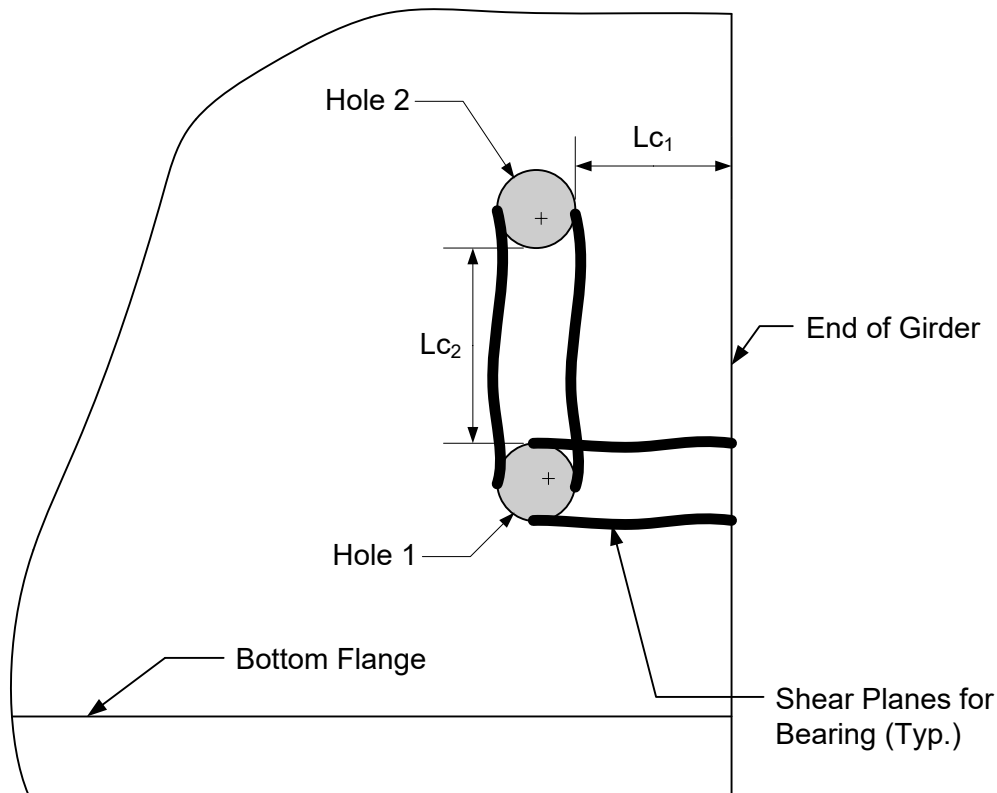
Figure 24.8-6
Bearing Resistance at Girder Web Bolt Holes

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 .
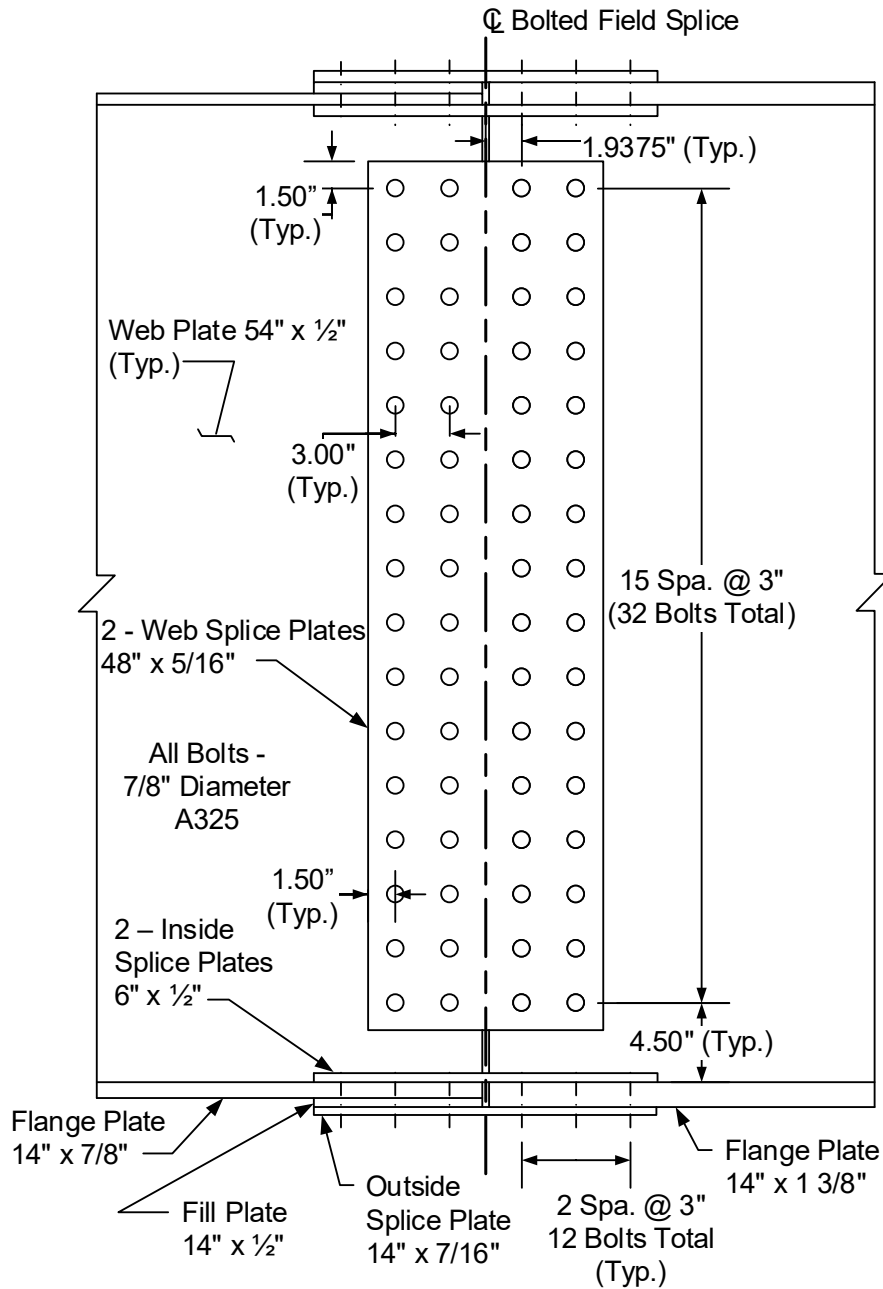
Figure 24.8-7
Sample Schematic of Final Splice Configuration

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:

$$b_t \le 0.48 t_p \sqrt{\frac{E}{F_{ys}}}$$

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.
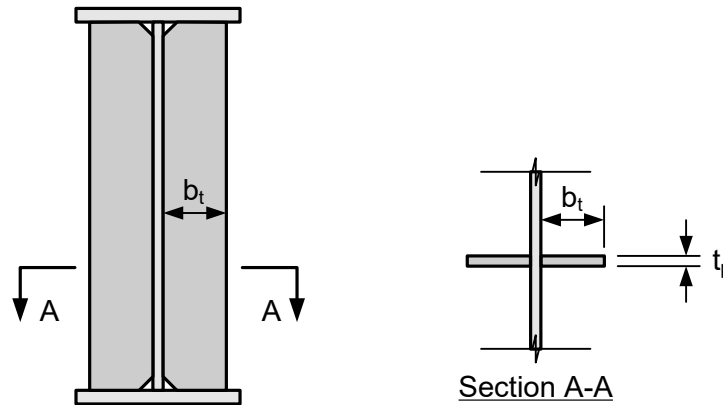
Figure 24.9-1
Projecting Width of a Bearing Stiffener

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:

$$\left(R_{sb}\right)_r = \phi_b \left(R_{sb}\right)_n$$

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:

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

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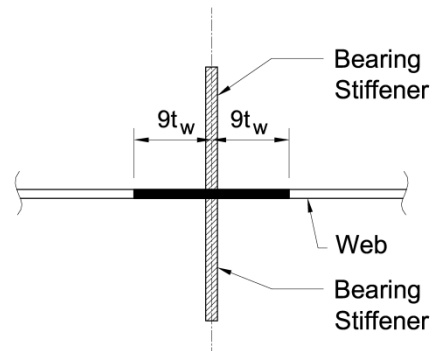
Figure 24.9-2
Effective Column Section for Welded Bearing Stiffener Design (One Pair of Stiffeners)

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*.

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E27-1 DESIGN EXAMPLE - STEEL REINFORCED ELASTOMERIC BEARING

This design example is for a 3-span prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.
(Example is current through LRFD Ninth Edition - 2020)

E27-1.1 Design Data

Bearing location: Abutment (Type A3)

Girder type: 72W

$L_{exp} := 220$ Expansion length, ft

$b_f := 2.5$ Bottom flange width, ft

$DL_{serv} := 167$ Service I limit state dead load, kips

$DL_{ws} := 23$ Service I limit state future wearing surface dead load, kips

$LL_{serv} := 62$ Service I limit state live load, kips

$h_{rcover} := 0.25$ Elastomer cover thickness, in

$h_s := 0.125$ Steel reinforcement thickness, in

$F_y := 36$ Minimum yield strength of the steel reinforcement, ksi

Temperature Zone:	D (Use for Entire State)	**LRFD [Fig. 14.7.5.2-1]**
Minimum Grade of Elastomer:	4	**LRFD [Table 14.7.5.2-1]**
Elastic Hardness:	Durometer 60 +/- 5	(used 55 for design)
Shear Modulus (G):	0.1125 ksi < G <0.165 ksi	**LRFD [Table 14.7.6.2-1]**
Creep Deflection @ 25 Years		
divided by instantaneous deflection:	0.3	**LRFD [Table 14.7.6.2-1]**

E27-1.2 Design Method

Use Design Method A **LRFD [14.7.6]**
Method A results in a bearing with a lower capacity than a bearing designed using Method B. However the increased capacity resulting from the use of Method B requires additional testing and quality control.

E27-1.3 Dynamic Load Allowance

The influence of impact need not be included for bearings **LRFD [14.4.1]**; however, dynamic load allowance will be included to follow a **WisDOT policy item**.

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30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

WisDOT policy item:

For all Interstate structures, the 42SS parapet shall be used. For all STH and USH structures with a posted speed >= 45 mph, the 42SS parapet shall be used.

The timeline for implementation of the above policy is:

- All contracts with a letting date after December 31, 2019.
  (This is an absolute, regardless of when the design was started.)

- All preliminary designs starting after October 1, 2017
  (Even if the let is anticipated to be prior to December 31, 2019.)

Contact BOS should the 42" height adversely affect sight distance, a minimum 0.5% grade for drainage cannot be achieved, or for other non-typical situations.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates

The application of bridge railings shall comply with the following guidance:

1. All bridge railings shall conform to **MASH 2016 requirements for lets after December 31, 2019**.

2. Traffic Railings placed on state-owned and maintained structures (Interstate Highways, United States Highways, State Trunk Highways, and roadways over such highways) with a design speed exceeding 45 mph shall be solid concrete parapets. Where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints, the designer shall utilize open railings as described in this section. (**NOTE**: WisDOT does not currently have an open rail meeting the minimum MASH TL-3 requirements for NHS roadways or non-NHS roadways with design speeds exceeding 45 mph. An open rail meeting MASH TL-3 is being investigated.).

   Traffic Railings placed on locally-owned and maintained structures (County Trunk Highways, Local Roadways) with a design speed exceeding 45 mph are strongly encouraged to utilize solid concrete parapets.

3. Traffic Railings placed on structures with a design speed of 45 mph or less can be either solid concrete parapets or open railings with the exception as noted below in the single slope parapet application section. It should be noted that open railing bridges can incur maintenance issues with salt-water runoff over the edge of deck.

4. New bridge plans utilizing concrete parapets shall be designed with single-sloped ("SS") parapets. See item No. 1 below for usage.

5. Per **LRFD [13.8.1]** and **LRFD [13.9.2]**, the minimum height of a Pedestrian (and/or bicycle) Railing shall be 42" measured from the top of the walkway or riding surface respectively. Per the *Wisconsin Bicycle Facility Design Handbook*, on bridges that are signed or marked as bikeways and bicyclists are operating right next to the railing, the preferred height of the railing is 54". The higher railing/parapet height is especially important and should be used on long bridges, high bridges, and bridges having high bicyclist volumes. If an open railing is used, the clear opening between horizontal elements shall be 6 inches or less.

6. Aesthetics associated with bridge railings shall follow guidance provided in 30.4.

7. For bridge railings on un-posted roadways, assume a design speed limit of 55 mph for determining the appropriate bridge railing.

The designation for railing types are shown on the Standard Details. Bridge railings shall be employed as follows:

1. <u>The default parapet shall be the "42SS".</u> If site distance issues arise due to the 42-inch height, please contact BOS for consideration of a shorter parapet ("32SS"and "36SS"). Single slope parapet "56SS" shall only be used if 56" CBSS adjoins the bridge. The "42SS" is TL-4 under MASH. The "32SS" is TL-3 under MASH. The "36SS" is TL-4 under MASH. *At this time, the "56SS" Test Loading is still unknown.*

A "SS" or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and "HF" parapets shall be used as Traffic Railings for rehabilitation projects (joint repair, impact damage, etc.) only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The "51F" parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. Although the vertical face parapet "A" can be used for all design speeds, Bureau of Structures Development Section approval is required for design speeds exceeding 45mph. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. The vertical face parapet "A" is considered at TL-3 when on a bridge deck and TL-2 when on a raised sidewalk (The structural capacity is TL-3, however the vaulting effect of the sidewalk lowers the rating to TL-2).

5. Aesthetic railings may be used if crash tested according to 30.1 or follow the guidance provided in 30.4. See Chapter 4 – Aesthetics for CSS considerations.

   The Texas style aesthetic parapet, type "TX", can be used as a Traffic/Pedestrian Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type "TX" parapet can be used. The type "TX" parapet is TL-2 under MASH.

6. The type "PF" tubular railing, as shown in the Standard Details of Chapter 40, shall <u>not</u> be used on new bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type "PF" railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type "C1" through "C6", are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5" from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. Combination railing, type "3T", without the recessed details on the parapet faces may be used when aesthetic details are not desired or when CSS funding is not available (see Chapter 4 – Aesthetics). These railings can only be used when the design speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete parapets to which they are attached.

8. Chain Link Fence and Tubular Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between the roadway and a sidewalk at grade. Due to snagging and breakaway potential of the vertical spindles, top-mounted Tubular Screening and Chain Link Fence should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

    Contact the Bureau of Structures Development Section when protective screening is warranted and used for design speeds exceeding 45 mph. In some cases, a Chain Link Fence mounted on the outside face (side-mounted) of the concrete parapet may be acceptable.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets ("A" or "SS") as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: "Type H (insert railing type) railing shall not be used". The combination railing is TL-3 under MASH.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing has not been rated under MASH.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type "W" railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. Although the type "W" railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 (based on a May 1997 FHWA memorandum), FHWA has since restricted its use as indicated above.

12. Type "M" steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type "M" railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type "M" railing also can be used in place of the type "W" railing when placed on girder type structures as type "W" railings are not allowed for this application. However, the type "M" railing is not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type "M" railing is TL-2 under MASH.

13. Type "NY3/NY4" steel railings, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type "NY3/NY4" railings may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type "NY3/NY4" railings also can be used in place of the type "W" railing when placed on girder type structures as type "W" railings are not allowed for this application. The type "NY4" railing may be used on a raised sidewalk where the design speed is 45 mph or less. However, the type "NY" railings are not allowed for use on prestressed box girder bridges. These railings shall be considered where the Region requests an open railing. The type "NY" railings are TL-2 under MASH.

14. The type "F" steel railing, as shown in the Standard Details of Chapter 40, shall <u>not</u> be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less. Details in Chapter 40 are for <u>informational purposes only</u>.

15. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the Facilities Development Manual (FDM) SDD 14b20. Railing is not required on box culverts if the culvert is extended to provide an adequate clear zone as defined in FDM 11-15-1. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. The railing shall be provided only when it is cost effective as defined in FDM 11-45-1.

16. When the structure approach thrie beam is extended across the box culvert; refer to Standard Detail, Box Culvert Details for additional information. The minimum dimension between end of box and face of guard rail provides an acceptable rail deflection to prevent a vehicle wheel from traversing over the end of the box culvert. In almost every case, the timber posts with offset blocks and standard beam guard are used. Type "W" railing may be used for maintenance and box culvert extensions to mitigate the effect of structure modifications.

See the FDM for additional railing application requirements. See FDM 11-45-1 and 11-45-2 for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See FDM 11-35-1 Table 1.2 for requirements when barrier wall separation between roadway and sidewalk is necessary.

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36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.12 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

36.3.3 Load Factors

In accordance with **LRFD [Table 3.4.1-1 and Table 3.4.1-2]**, the following Strength I load factors, γ_{st}, and Service I load factors, γ_{s1}, shall be used for box culvert design:

		Strength I Load Factor, γ_{st}		Service I Load Factor, γ_{s1}
Type of Load		Max.	Min.	
Dead Load-Components	DC	1.25	0.90	1.0
Dead Load-Wearing Surface	DW	1.50	0.65	1.0
Vertical Earth Pressure	EV	1.35	0.90	1.0
Horizontal Earth Pressure	EH	1.35	0.50[1]	1.0
Live Load Surcharge	LS	1.75	1.75	1.0
Live Load + IM	LL+IM	1.75	1.75	1.0

[1]Per **LRFD [3.11.7]**, for culverts where earth pressure may reduce effects caused by other loads, a 50% reduction may be used, but not combined with the minimum load factor specified in **LRFD [Table 3.4.1-2]**.

36.3.4 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a structure is expected to experience during its design life **LRFD [1.3.2.4]**.

36.3.4.1 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for the variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for reinforced concrete box culverts for the Strength Limit State per **LRFD [Table 12.5.5-1]** are as shown below:

Structure Type	Flexure	Shear
Cast-In-Place	0.90	0.85
Precast	1.00	0.90
Three-Sided	0.95	0.90

36.3.4.2 Moment Capacity

For rectangular sections, the nominal moment resistance, M_n, per **LRFD [5.6.3.2.3]** (tension reinforcement only) equals:

$$M_n = A_s f_s \left(d_s - \frac{a}{2}\right)$$

The factored resistance, M_r, or moment capacity per **LRFD [5.6.3.2.1]**, shall be taken as:

$$M_r = \phi M_n = \phi A_s f_s \left(d_s - \frac{a}{2}\right)$$

For additional information on concrete moment capacity, including stress and strain assumptions used, refer to 18.3.3.2.1.

The location of the design moment will consider the haunch dimensions in accordance with **LRFD [12.11.5.2]**. No portion of the haunch shall be considered in adding to the effective depth of the section.

36.3.4.3 Shear Capacity

Per **LRFD [12.11.5.1]**, shear in culverts shall be investigated in conformance with **LRFD [5.12.7.3]**. The location of the critical section for shear for culverts with haunches shall be determined in conformance with **LRFD [C5.12.8.6.1]** and shall be taken at a distance d_v from the end of the haunch.

36.3.4.3.1 Depth of Fill Greater than or Equal to 2.0 ft.

The shear resistance of the concrete, V_c, for <u>slabs</u> of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.12.7.3]** shall be determined as:

$$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \leq 0.126\lambda\sqrt{f'_c}\,bd_e$$

Where:

$$\frac{V_u d_e}{M_u} \leq 1$$

Where:

V_c	=	Shear resistance of the concrete (kip)
A_s	=	Area of reinforcing steel in the design width (in^2)

d_e	=	Effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)
V_u	=	Factored shear (kip)
M_u	=	Factored moment, occurring simultaneously with V_u (kip-in)
b	=	Design width (in.)
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

In the absence of shear reinforcing, the nominal shear resistance is equal to the shear resistance of the concrete. The factored resistance, V_r, or shear capacity, per **LRFD [5.7.2.1]** shall be taken as:

$$V_r = \phi V_n = \phi V_c$$

Per **LRFD [5.12.7.3]**, for single-cell box culverts only, Vc for slabs monolithic with walls need not be taken less than:

$$0.0948 \cdot \lambda \sqrt{f'_c}\, bd_e$$

and V_c for slabs simply supported need not be taken less than:

$$0.0791 \cdot \lambda \sqrt{f'_c}\, bd_e$$

The shear resistance of the concrete, V_c, for <u>walls</u> of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.7.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta \lambda \sqrt{f'_c}\, b_v d_v \leq 0.25 f'_c\, b_v d_v$$

Where:

V_c	=	Shear resistance of the concrete (kip)
β	=	2.0 (**LRFD [5.7.3.4.1]**)
b_v	=	Effective web width taken as the minimum web width within the depth d_v (in.)
d_v	=	Effective shear depth as determined in **LRFD [5.7.2.8]**. Perpendicular distance between tension and compression resultants. Need not be taken less than the greater of $0.9d_e$ or 0.72h (in.)
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

36.3.4.3.2 Depth of Fill Less than 2.0 ft

Per **LRFD [5.12.7.3]**, for box culverts with less than 2.0 feet of fill follow **LRFD [5.7]** and **LRFD [5.12.8.6]**.

The shear resistance of the concrete, V_c, for <u>slabs and walls</u> of box culverts with less than 2.0 feet of fill, for one-way action per **LRFD [5.7.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta\lambda\sqrt{f'_c}\, b_v d_v \le 0.25 f'_c\, b_v d_v$$

With variables defined above in 36.3.4.3.1.

For box culverts where the top slab is an integral part of the wearing surface (depth of fill equal zero) the top slab shall be checked for two-way action, as discussed in 18.3.3.2.2.

36.3.5 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**.

36.3.5.1 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

36.3.5.2 Crack Control Criteria

Per **LRFD [12.11.4]**, the provisions of **LRFD [5.6.7]** shall apply to crack width control in box culverts. All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r, specified in **LRFD [5.4.2.6]** for Service I Limit State. The spacing, s, (in inches) of mild steel reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(in.)}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

γ_e	=	Exposure factor (1.0 for Class 1 exposure condition, 0.75 for Class 2 exposure condition, see **LRFD [5.6.7]** for guidance)
d_c	=	Thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.). For top slab reinforcement with no fill, d_c, should not include the ½" wearing surface
f_{ss}	=	Tensile stress in steel reinforcement at the service limit state (ksi) ≤ 0.6 f_y
h	=	Overall thickness or depth of the component (in.)

WisDOT Policy Item:

A class 1 exposure factor, γ_e = 1.0, shall be used for all cases for cast-in-place box culverts except for the top steel in the top slab of a box culvert with zero fill, where a class 2 exposure factor, γ_e = 0.75, shall be used.

36.3.6 Minimum Reinforcement Check

Per **LRFD [12.11.5.3]**, the area of reinforcement, A_s, in the box culvert cross-section should be checked for minimum reinforcement requirements per **LRFD [5.6.3.3]**.

The area of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, or moment capacity at least equal to the lesser of:

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

γ_1	=	1.6 flexural cracking variability factor
γ_3	=	0.67 ratio of minimum yield strength to ultimate tensile strength; for <u>A615 Grade 60 reinforcement</u>
f_r	=	$0.24\lambda\sqrt{f'_c}$ Modulus of rupture (ksi) **LRFD [5.4.2.6]**
I_g	=	Gross moment of inertia (in^4)

c	=	½ *effective slab thickness (in.)

M_u = Total factored moment using Strength I Limit State (kip-in)

M_{cr} = Cracking strength moment (kip-in)

λ = concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

The factored resistance, M_r or moment capacity, shall be calculated as in 36.3.4.2 and shall satisfy:

$$M_r \geq \min (M_{cr} \text{ or } 1.33 \ M_u)$$

36.3.7 Minimum Spacing of Reinforcement

Per **LRFD [5.10.3.1]**, the clear distance between parallel bars in a layer shall not be less than:

- 1.5 times the nominal diameter of the bars

- 1.5 times the maximum size of the course aggregate

- 1.5 inches

36.3.8 Maximum Spacing of Reinforcement

Per **LRFD [5.10.3.2]**, the spacing of reinforcement in walls and slabs shall not exceed:

- 1.5 times the thickness of the member (3.0 times for temperature and shrinkage)

- 18 inches

36.3.9 Edge Beams

Per **LRFD [12.11.2.1]**, for cast-in-place box culverts, and for precast box culverts with top slabs having span to thickness ratios (s/t) > 18 or segment lengths < 4.0 feet, edge beams shall be provided as specified in **LRFD [4.6.2.1.4]** as follows:

- At ends of culvert runs where wheel loads travel within 24.0 inches from the end of the culvert

- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint

The edge beam provisions are only applicable for culverts with less than 2.0 ft of fill, **LRFD [C12.11.2.1]**.

Where:

W_t	=	Factored earth pressure on top of box culvert (ksf)
γ_{stEV}	=	Vertical earth pressure load factor
γ_{stEH}	=	Horizontal earth pressure load factor
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)

Figure 36.4-1 shows the factored vertical and horizontal earth load pressures acting on a box culvert. The soil pressure on the bottom of the box is not shown, but shall be determined for the design of the bottom slab. Note: vertical earth pressures, as well as other loads (e.g. DC and DW), are typically distributed equally over the bottom of the box when determining pressure distributions for the bottom slab. Pressure distributions from a refined analysis is typically not warranted for new culvert designs, but should be considered when evaluating existing culvert sections on culvert extension projects.

36.4.4 Live Load Surcharge (LS)

Per **LRFD [3.11.6.4]**, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the distance from top of pavement to bottom of the box culvert.

Per **LRFD [Table 3.11.6.4-1]**, the following equivalent heights of soil for vehicular loading shall be used. The height to be used in the table shall be taken as the distance from the bottom of the culvert to the roadway surface. Use interpolation for heights other than those listed in the table.

Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Table 36.4-1
Equivalent Height of Soil for Vehicular Loading

Surcharge loads are computed based on a coefficient of lateral earth pressure times the unit weight of soil times the height of surcharge. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil, as discussed in 36.4.3. The uniform distributed load is applied to both exterior walls with the load directed toward the center of the box culvert. The load is designated as, LS, live load surcharge, for application of load factors and limit state combinations. Refer to **LRFD [3.11.6.4]** for additional information regarding live load surcharge.

36.4.5 Water Pressure (WA)

Static water pressure loads are computed when the water height on the outside of the box is greater than zero. The water height is measured from the bottom inside of the box culvert to the water level. The load is designated as, WA, water pressure load, for application of load factors and limit state combinations. Water pressure in culvert barrels is ignored. Refer to **LRFD [3.7.1]** for additional information regarding water pressure.

36.4.6 Live Loads (LL)

Live load consists of the standard AASHTO LRFD trucks and tandem. Per **LRFD [3.6.1.3.3]**, design loads are always axle loads (single wheel loads should not be considered) and the lane load is not used. The depth of fill is measured from top of culvert to surface of earth fill or top roadway pavement.

Where the depth of fill over the box is less than 2 feet, the wheel loads are distributed per **LRFD [4.6.2.10]**. Where the depth of fill is 2 feet or more, the wheel loads shall be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area **LRFD [3.6.1.2.5]**, increased by the live load distribution factor (LLDF) in **LRFD [Table 3.6.1.2.6a-1]**, using the provisions of **LRFD [3.6.1.2.6b-c]**. Where areas from distributed wheel loads overlap at the top of the culvert, the total load is considered as uniformly distributed over the rectangular area (A_{LL}) defined by the outside limits described in **LRFD [3.6.1.2.6b-c]**.

Per **LRFD [3.6.1.2.6a]**, for single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between inside faces of end walls. **LRFD [3.6.1.2.6a]** also states, if designing a culvert with fill of 2 feet or more, calculate live load design moments using the method in **LRFD [3.6.1.2.6b-c]** and also calculate live load design moments using the method in **LRFD [4.6.2.10]**. Then select and use the method that provides the smaller live load design moments.

Skew is not considered for design loads.

36.4.6.1 Depth of Fill Less than 2.0 ft.

Where the depth of fill is less than 2.0 ft, follow **LRFD [4.6.2.10]**.

36.4.6.1.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [4.6.2.10.2]**. Use a single lane and the single lane multiple presence factor of 1.2.

Distribution length perpendicular to the span:

$$E = (96 + 1.44(S))$$

Where:

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E36-1 Twin Cell Box Culvert LRFD

This example shows design calculations for a twin cell box culvert. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Ninth Edition - 2020)***

E36-1.1 Design Criteria
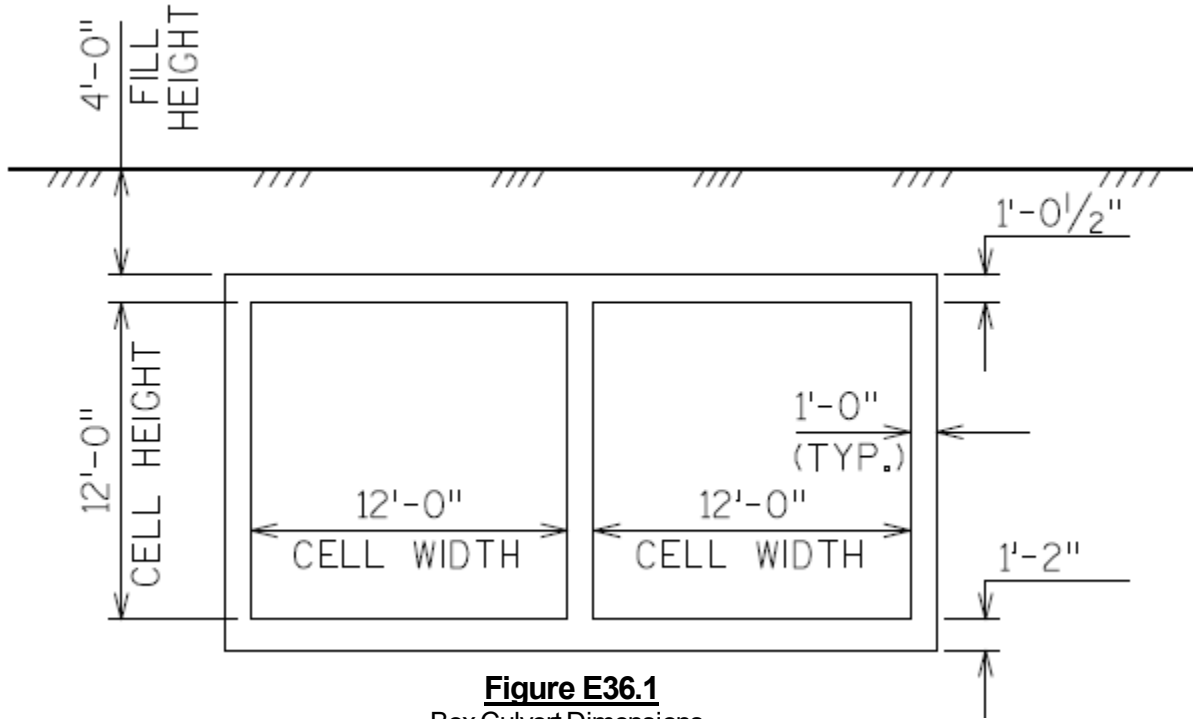
Figure E36.1
Box Culvert Dimensions

$NC = 2$	number of cells
$Ht = 12.0$	cell height, ft
$W_1 = 12.0$	cell 1 width, ft
$W_2 = 12.0$	cell 2 width, ft
$L = 134.0$	culvert length, ft
$t_{ts} = 12.5$	top slab thickness, in
$t_{bs} = 14.0$	bottom slab thickness, in
$t_{win} = 12.0$	interior wall thickness, in
$t_{wex} = 12.0$	exterior wall thickness, in

$$H_{apron} := Ht + \frac{t_{ts}}{12} \qquad \text{apron wall height above floor, ft}$$

$$H_{apron} = 13.04 \quad \text{ft.}$$

E36-1.7 Design Reinforcement Bars

Design of the corner bars is illustrated below. Calculations for bars in other locations are similar.

Design Criteria:

For corner bars, use the controlling thickness between the slab and wall. The height of the concrete design section is:

$$h := \min\left(t_{ts}, t_{bs}, t_{wex}\right)$$

$\boxed{h = 12.00}$ in

Use a 1'-0" design width:

$b := 12.0$ width of the concrete design section, in

$cover = 2.0$ concrete cover, in   Note: The calculations here use 2" cover for the top slab and walls. Use 3" cover for the bottom of the bottom slab (not shown here).

$Mstr1_{CB} = 16.73$ design strength moment, kip-ft

$Ms1_{CB} = 11.18$ design service moment, kip-ft

$f_s := f_y$ reinforcement yield strength, ksi $f_y = 60.00$ ksi

$Bar_{No} := 5$ assume #5 bars (for d_s calculation)

$Bar_D\left(Bar_{No}\right) = 0.63$ bar diameter, in

Calculate the estimated distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement. **LRFD [5.6.3.2.2]**

$$d_{s\_i} := h - cover - \frac{Bar_D\left(Bar_{No}\right)}{2}$$

$\boxed{d_{s\_i} = 9.69}$ in

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ per **LRFD [Table 12.5.5-1]**.

Calculate the coefficient of resistance:

$$R_n := \frac{Mstr1_{CB} \cdot 12}{\phi_f \cdot b \cdot d_{s\_i}^2}$$

$\boxed{R_n = 0.20}$ ksi

Calculate the reinforcement ratio:

$$\rho := 0.85 \cdot \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1.0 - \frac{2 \cdot R_n}{0.85 \cdot f'_c}}\right)$$

$\boxed{\rho = 0.0034}$

Calculate the required area of steel:

$$A_{s\_req'd} := \rho \cdot b \cdot d_{s\_i}$$

$\boxed{A_{s\_req'd} = 0.40}$ in²

Given the required area of steel of $A_{s\_req'd} = 0.40$, try #5 bars at 7.5" spacing:

$\boxed{Bar_{No} := 5}$ bar size

$\boxed{spacing := 7.0}$ bar spacing, in

The area of one reinforcing bar is:

$$A_{s\_1bar} := Bar_A(Bar_{No})$$

$\boxed{A_{s\_1bar} = 0.31}$ in²

Calculate the area of steel in a 1'-0" width

$$A_s := \frac{A_{s\_1bar}}{\dfrac{spacing}{12}}$$

$\boxed{A_s = 0.53}$ in²

Check that the area of steel provided is larger than the required area of steel

Is $A_s = 0.53$ in² $\geq A_{s\_req'd} = 0.40$ in²

$\boxed{check = "OK"}$

Recalculate d_c and d_s based on the actual bar size used.

$$d_c := cover + \frac{Bar_D(Bar_{No})}{2}$$

$\boxed{d_c = 2.31}$ in

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$

$\boxed{d_s = 9.69}$ in

Per **LRFD [5.6.2.2]**, The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65. The factor α_1 shall be taken as 0.85 for concrete strength not exceeding 10.0 ksi.

$\boxed{\beta_1 = 0.85}$ $\boxed{\alpha_1 = 0.85}$

Per **LRFD [5.6.2.1]**, if $\dfrac{c}{d_s} \leq 0.6$ (for $f_y = 60$ ksi) then reinforcement has yielded and the assumption is correct.

"c" is defined as the distance between the neutral axis and the compression face (inches).

$$c := \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$

$\boxed{c = 1.05}$ in

Check that the reinforcement will yield:

$\boxed{check = "OK"}$

Is $\dfrac{c}{d_s} = 0.11 \leq 0.6$?

therefore, the reinforcement will yield

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.6.3.2.3]**:

$$a := \beta_1 \cdot c$$

$\boxed{a = 0.89}$ in

$$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2} \right) \frac{1}{12} \right]$$

$\boxed{M_n = 24.6}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$$M_r := \phi_f \cdot M_n$$

$\boxed{M_r = 22.1}$ kip-ft

The required capacity:

Corner Moment $\boxed{\text{Mstr1}_{CB} = 16.7}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.6.3.3]**:

$b = 12.0$ in   width of the concrete design section, in

$h = 12.0$ in   height of the concrete design section, in

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]** $f_r = 0.45$ ksi

$$I_g := \frac{1}{12} \cdot b \cdot h^3$$ gross moment of inertia, in⁴ $I_g = 1728.00$ in⁴

$$\frac{h}{2} = 6.0$$ distance from the neutral axis to the extreme element

$$S_c := \frac{I_g}{\frac{h}{2}}$$ section modulus, in³ $S_c = 288.00$ in³

The corresponding cracking moment is:

$$M_{cr} = \gamma_3 (\gamma_1 \cdot f_r) S_c$$ therefore, $M_{cr} = 1.1 (f_r) S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement
for A615, Grade 60 reinforcement

$$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$$

$\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot \text{Mstr1}_{CB} = 22.2}$ kip-ft

Is $M_r = 22.1$ kip-ft greater than the lesser of M_{cr} and $1.33 \cdot M_{str}$?   ⟦check = "OK"⟧

Per **LRFD [5.6.7]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.0$ for Class 1 exposure condition

$h = 12.0$ height of the concrete design section, in

Calculate the ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face:

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad\qquad \boxed{\beta_s = 1.34}$$

Calculate the reinforcement ratio:

$$\rho := \frac{A_s}{b \cdot d_s} \qquad\qquad \boxed{\rho = 0.0046}$$

Calculate the modular ratio:

$$N := \frac{E_s}{E_c} \qquad\qquad \boxed{N = 8.06}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi). The moment arm used in the equation below to calculate f_{ss} is: (j) (h-d_c)

$$k := \sqrt{(\rho \cdot N)^2 + (2 \cdot \rho \cdot N)} - \rho \cdot N \qquad\qquad \boxed{k = 0.2370}$$

$$j := 1 - \frac{k}{3} \qquad\qquad\qquad\qquad \boxed{j = 0.9210}$$

$Ms1_{CB} = 11.18$ service moment, kip-ft

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \le 0.6\,f_y \qquad\qquad \boxed{f_{ss} = 28.29} \quad \text{ksi} \quad \le 0.6\,f_y \text{ O.K.}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad\qquad s_{max1} = 13.83 \quad \text{in}$$

$$s_{max2} := min(1.5 \cdot h, 18) \qquad\qquad s_{max2} = 18.00 \quad \text{in}$$

$$s_{max} := min(s_{max1}, s_{max2}) \qquad\qquad \boxed{s_{max} = 13.83} \quad \text{in}$$

Check that the provided spacing is less than the maximum allowable spacing

Is spacing $= 7.00$ in $\le s_{max} = 13.83$ in $\boxed{\text{check} = \text{"OK"}}$

Calculate the minimum spacing requirements per **LRFD [5.10.3.1]**. The clear distance between parallel bars in a layer shall not be less than:

$$S_{min1} := 1.5 \cdot Bar_D(Bar_{No}) \qquad\qquad S_{min1} = 0.94 \quad \text{in}$$

$$S_{min2} := 1.5 \cdot 1.5 \quad \text{(maximum aggregate size = 1.5 inches)} \qquad S_{min2} = 2.25 \quad \text{in}$$

$$S_{min3} := 1.5 \quad \text{in}$$

Is spacing $= 7.00$ in $\ge$ all minimum spacing requirements? $\boxed{\text{check} = \text{"OK"}}$

E36-1.8 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceding sections.

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.6]**

$$A_s \ge \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \qquad \text{and} \qquad 0.11 \le A_s \le 0.60$$

Where:

A_s = area of reinforcement in each direction and each face $\left(\dfrac{in^2}{ft} \right)$

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars (ksi) ≤ 75 ksi

Check the minimum required temperature and shrinkage reinforcement, #4 bars at 15", in the thickest section. For the given cross section, the values for the corner bar design are:

$$A_{s\_4\_at\_15} := \frac{Bar_A(4)}{1.25} \qquad\qquad \boxed{A_{s\_4\_at\_15} = 0.16} \quad \frac{in^2}{ft}$$

$$b_{TS} := \max\left(t_{ts}, t_{bs}, t_{wex}\right)$$

$\boxed{b_{TS} = 14.0}$ in

$$h_{TS} := 12\left(W_1 + W_2\right) + 2 \cdot t_{wex} + t_{win}$$

$\boxed{h_{TS} = 324.0}$ in

$$f_y = 60.00 \quad \text{ksi}$$

For each face, the required area of steel is:

$$A_{s\_TS} := \frac{1.30 \cdot \left(b_{TS}\right) \cdot h_{TS}}{2 \cdot \left(b_{TS} + h_{TS}\right) \cdot f_y}$$

$A_{s\_TS} = 0.15 \quad \dfrac{\text{in}^2}{\text{ft}}$

is $A_{s\_4\_at\_15} = 0.16$ in2 $\geq A_{s\_TS} = 0.15$ in² ?

$\boxed{\text{check} = \text{"OK"}}$

is $0.11 < A_{s\_4\_at\_15} < 0.60$?

$\boxed{\text{check} = \text{"OK"}}$

Per **LRFD [5.10.6]**, the shrinkage and temperature reinforcement shall not be spaced farther apart than:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in for walls and footings greater than 18.0 in. thick
- 12.0 in for other components greater than 36.0 in. thick

$$s_{max3} = 18.00 \quad \text{in}$$

Per **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the member or 18.0 in.

$$s_{max4} = 18.00 \quad \text{in}$$

is the 15" spacing $\leq$ both maximum spacing requirements?

$\boxed{\text{check} = \text{"OK"}}$

Note: The design of the bottom slab shrinkage and temperature bars is illistrated above. Shrinkage and temperature bars may be reduced or not required at other locations. See Section 36.6.8 and Standard 36.03 for additional information.

The results for the other bar locations are shown in the table below:

Results						
Location	ΦMn	$A_{S\ Req'd}$	$A_{S\ Actual}$	Bar Size	S_{max}	S_{actual}
Corner	22.1	0.48	0.53	5	13.8	7.0
Pos. Mom. Top Slab	21.8	0.49	0.50	5	13.0	7.5
Pos. Mom. Bot. Slab	28.9	0.54	0.57	5	18.0	6.5
Neg. Mom. Top Slab	23.3	0.50	0.53	5	12.1	7.0
Neg. Mom. Bot. Slab	28.4	0.54	0.62	5	13.4	6.0
Exterior Wall	16.9	0.34	0.40	4	18.0	6.0
Interior Wall	6.9	0.15	0.16	4	18.0	15.0

E36-1.9 Distribution Reinforcement

Per **LRFD [9.7.3.2]**, reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

Distribution steel is not required when the depth of fill over the slab exceeds 2 feet, **LRFD [5.12.2.1]**.

E36-1.10 Reinforcement Details

The reinforcement bar size and spacing required from the strength and serviceability calcuations above are shown below:
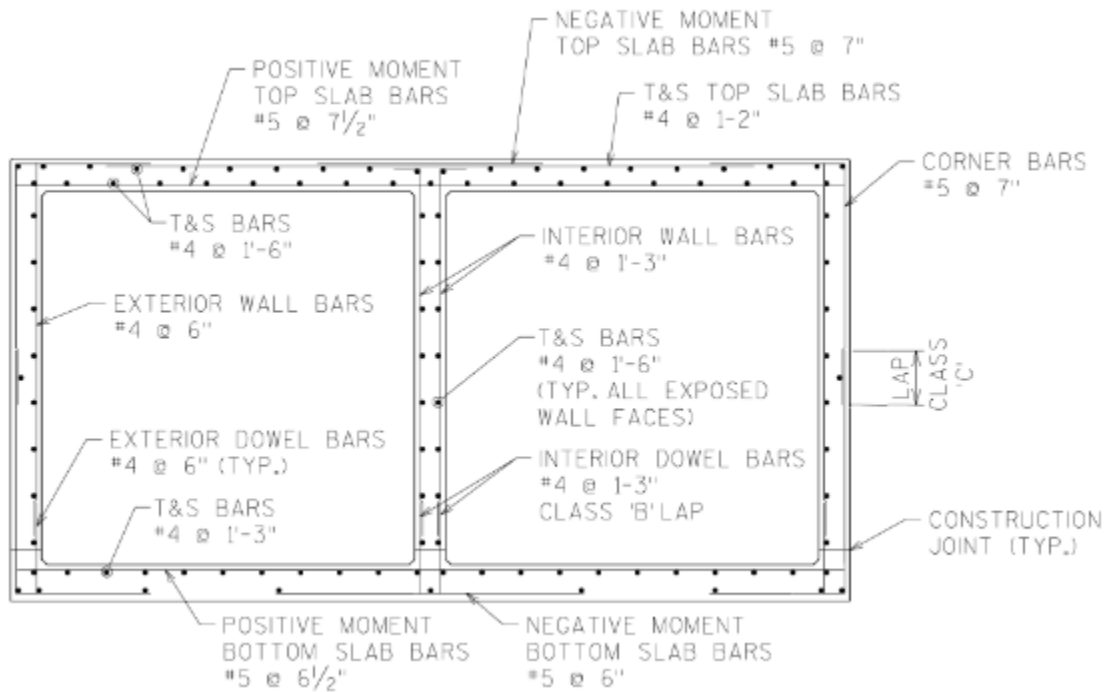
E36-1.11 Cutoff Locations

Determine the cutoff locations for the corner bars. Per Sect. 36.6.1, the distance "L" is computed from the maximum negative moment envelope for the top slab.

The cutoff lengths are in feet, measured from the inside face of the exterior wall.

Initial Cutoff Locations:

The initial cutoff locations are determined from the inflection points of the moment diagrams.

Corner Bars	$\text{CutOff1}_{CBH\_i} = 2.64$	$\text{CutOff2}_{CBH\_i} = 1.15$ Horizontal
		$\text{CutOff2}_{CBV\_i} = 2.07$ Vertical
Positive Moment Top Slab Bars	$\text{CutOff1}_{PTS\_i} = 1.26$	$\text{CutOff2}_{PTS\_i} = 1.86$
Positive Moment Bottom Slab Bars	$\text{CutOff1}_{PBS\_i} = 1.27$	$\text{CutOff2}_{PBS\_i} = 1.97$
Negative Moment Top Slab Bars	$\text{CutOff1}_{NTS\_i} = 8.63$	$\text{CutOff2}_{NTS\_i} = 10.32$
Negative Moment Bottom Slab Bars	$\text{CutOff1}_{NBS\_i} = 8.97$	$\text{CutOff2}_{NBS\_i} = 10.56$

For the second cutoff location for each component, the following checks shall be completed:

Check the section for minimum reinforcement in accordance with **LRFD [5.6.3.3]**:

The required capacity at the second cutoff location (for the vertical leg of the corner bar):

$\text{Mstr1}_{CBV2} = 7.89$ strength moment at the second cutoff location, kip-ft

The usable capacity of the remaining bars is calculated as follows:

$$A_{s2} := \frac{A_s}{2} \qquad\qquad \boxed{A_{s2} = 0.27} \text{ in}^2$$

$$c2 := \frac{A_{s2} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \qquad \boxed{\beta_1 = 0.85} \; \boxed{\alpha_1 = 0.85} \qquad \boxed{c2 = 0.53} \text{ in}$$

$$a2 := \beta_1 \cdot c2 \qquad\qquad\qquad\qquad\qquad\qquad \boxed{a2 = 0.45} \text{ in}$$

$$M_{n2} := \left[A_{s2} \cdot f_s \cdot \left(d_s - \frac{a2}{2} \right) \frac{1}{12} \right] \qquad \boxed{M_{n2} = 12.6} \text{ kip-ft}$$

$$M_{r2} := \phi_f \cdot M_{n2} \qquad\qquad\qquad\qquad \boxed{M_{r2} = 11.3} \text{ kip-ft}$$

Is $M_{r2} = 11.3$ kip-ft greater than the lesser of M_{cr} and $1.33*M_{str}$? $\boxed{\text{check} = \text{"OK"}}$

$$\boxed{M_{cr} = 11.9} \qquad \text{kip-ft}$$

$$\boxed{1.33 \cdot \text{Mstr1}_{CBV2} = 10.5} \quad \text{kip-ft}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi).

$$\text{Ms1}_{CBV2} = 3.43 \qquad \text{service moment at the second cutoff location, kip-ft}$$

$$f_{ss2} := \frac{\text{Ms1}_{CBV2} \cdot 12}{A_{s2} \cdot (j) \cdot (h - d_c)} \qquad \boxed{f_{ss2} = 17.35} \quad \text{ksi}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max2\_1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss2}} - 2 \cdot d_c \qquad\qquad s_{max2\_1} = 25.47 \quad \text{in}$$

$$s_{max2\_2} := s_{max2} \qquad\qquad s_{max2\_2} = 18.00 \quad \text{in}$$

$$s_{max} := \min\left(s_{max2\_1}, s_{max2\_2}\right) \qquad \boxed{s_{max} = 18.00} \qquad \text{in}$$

Check that the provided spacing (for half of the bars) is less than the maximum allowable spacing

$$\text{spacing2} := 2 \cdot \text{spacing} \qquad\qquad \text{spacing2} = 14.00 \quad \text{in}$$

Is spacing2 $= 14.00$ in $\leq$ $s_{max} = 18.00$ in $\boxed{\text{check} = \text{"OK"}}$

Extension Lengths:

The extension lengths for the corner bars are shown below. Calculations for other bars are similar.

Extension lengths for general reinforcement per **LRFD [5.10.8.1.2a]**:

$$\text{MaxDepth} := \max\!\left(t_{ts} - \text{cover}, t_{wex} - \text{cover}, t_{bs} - \text{cover}_{bot}\right) \qquad \text{MaxDepth} = 11.00 \quad \text{in}$$

Effective member depth
$$\frac{\text{MaxDepth} - \dfrac{1}{2}\,\text{Bar}_D\!\left(\text{Bar}_{No\_CB}\right)}{12} = 0.89 \quad \text{ft}$$

15 x bar diameter
$$\frac{15 \cdot \text{Bar}_D\!\left(\text{Bar}_{No\_CB}\right)}{12} = 0.78 \quad \text{ft}$$

1/20 times clear span
$$\frac{\max\!\left(W_1, W_2\right)}{20} = 0.60 \quad \text{ft}$$

The maximum of the values listed above:

$$\text{ExtendLength\_gen}_{CB} = 0.89 \qquad \text{ft}$$

Extension lengths for negative moment reinforcement per **LRFD [5.10.8.1.2c]**:

Effective member depth
$$\frac{\text{MaxDepth} - \dfrac{1}{2}\,\text{Bar}_D\!\left(\text{Bar}_{No\_CB}\right)}{12} = 0.89 \quad \text{ft}$$

12 x bar diameter
$$\frac{12 \cdot \text{Bar}_D\!\left(\text{Bar}_{No\_CB}\right)}{12} = 0.63 \quad \text{ft}$$

0.0625 times clear span
$$0.0625 \max\!\left(W_1, W_2\right) = 0.75 \quad \text{ft}$$

The maximum of the values listed above:

$$\text{ExtendLength\_neg}_{CB} = 0.89 \qquad \text{ft}$$

The development length:

$$\text{DevLength}_{CB} = 1.00 \quad \text{ft}$$

The extension lengths for general reinforcment for the other bars are:

Corner Bars $\text{ExtendLength\_gen}_{CB} = 0.89$ ft

Positive Moment Top Slab Bars $\text{ExtendLength\_gen}_{PTS} = 0.85$ ft

Positive Moment Bottom Slab Bars $\text{ExtendLength\_gen}_{PBS} = 0.97$ ft

Negative Moment Top Slab Bars $\text{ExtendLength\_gen}_{NTS} = 0.85$ ft

Negative Moment Bottom Slab Bars $\text{ExtendLength\_gen}_{NBS} = 0.97$ ft

The extension lengths for negative moment reinforcment for the other bars are:

Corner Bars $\text{ExtendLength\_neg}_{CB} = 0.89$ ft

Positive Moment Top Slab Bars $\text{ExtendLength\_neg}_{PTS} = 0.85$ ft

Positive Moment Bottom Slab Bars $\text{ExtendLength\_neg}_{PBS} = 0.97$ ft

Negative Moment Top Slab Bars $\text{ExtendLength\_neg}_{NTS} = 0.85$ ft

Negative Moment Bottom Slab Bars $\text{ExtendLength\_neg}_{NBS} = 0.97$ ft

The final cutoff locations (measured from the inside face of the exterior wall) are:

Corner Bars	$CutOff1_{CBH} = 3.53$	$CutOff2_{CBH} = 2.04$	Horizontal
		$CutOff2_{CBV} = 2.96$	Vertical

Positive Moment Top Slab Bars $CutOff1_{PTS}$ = "Run Bar Entire Width of Box"

$CutOff2_{PTS} = 1.02$

Positive Moment Bottom Slab Bars $CutOff1_{PBS}$ = "Run Bar Entire Width of Box"

$CutOff2_{PBS} = 1.00$

Negative Moment Top Slab Bars	$CutOff1_{NTS} = 7.78$	$CutOff2_{NTS} = 9.47$
Negative Moment Bottom Slab Bars	$CutOff1_{NBS} = 7.99$	$CutOff2_{NBS} = 9.59$

The cutoff locations for the corner bars are shown below. Other bars are similar.
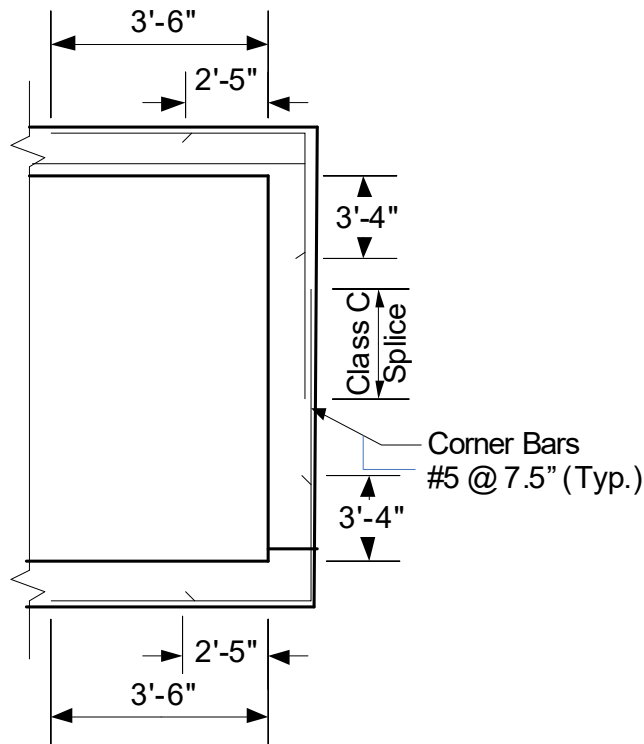
E36-1.12 Shear Analysis

Analyze walls and slabs for shear

E36-1.12.1 Factored Shears

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored shears for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

<u>Strength 1 Shears</u>

$$V_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot V_{DC} + \gamma st_{DW} \cdot V_{DW} + \gamma st_{EV} \cdot V_{EV} + \gamma st_{EH} \cdot V_{EH} + \gamma st_{LS} \cdot V_{LS} + \gamma st_{LL} \cdot V_{LL} \right)$$

Exterior Wall	$Vstr1_{XW} = 7.98$	kip
Interior Wall	$Vstr1_{IW} = 0.40$	kip
Top Slab	$Vstr1_{TS} = 12.20$	kip
Bottom Slab	$Vstr1_{BS} = 12.16$	kip

<u>Service 1 Shears</u>

$$V_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot V_{DC} + \gamma s1_{DW} \cdot V_{DW} + \gamma s1_{EV} \cdot V_{EV} + \gamma s1_{EH} \cdot V_{EH} + \gamma s1_{LS} \cdot V_{LS} + \gamma s1_{LL} \cdot V_{LL} \right)$$

Exterior Wall	$Vs1_{XW} = 5.64$	kip
Interior Wall	$Vs1_{IW} = 0.23$	kip
Top Slab	$Vs1_{TS} = 7.62$	kip
Bottom Slab	$Vs1_{BS} = 7.96$	kip

E36-1.12.2 Concrete Shear Resistance

Check that the nominal shear resistance, V_n, of the concrete in the top slab is adequate for shear without shear reinforcement per **LRFD [5.12.7.3]**.

$$V_n = V_c = \left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u} \right) \cdot b \cdot d_s \leq 0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s$$

$f'_c = 3.5$　　　　　　culvert concrete strength, ksi

$A_{s\_TS} = 0.15$　　　　　area of reinforcing steel in the design width, in²/ft width

$h := t_{ts}$　　　　　height of concrete design section, in　　　　$h = 12.50$　in

$\lambda = 1.0$　　　　　normal wgt. conc. **LRFD [5.4.2.8]**

Calculate d_s, the distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$

$\boxed{d_s = 10.19}$　in

$$V_u := Vstr1_{TS}$$

$\boxed{V_u = 12.2}$　kips

$M_u = 264.01$　　　　factored moment occurring simultaneously with V_u, kip-in

$b := 12$　　　　　design width, in

For reinforced concrete cast-in-place box structures, $\phi_V = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$\dfrac{V_u \cdot d_s}{M_u}$　shall not be taken to be greater than 1.0　　　　　$\dfrac{V_u \cdot d_s}{M_u} = 0.47$　　< 1.0 OK

$$V_{r1s} := \phi_v \cdot \left[\left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_{s\_TS}}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u} \right) \cdot b \cdot d_s \right]$$

$\boxed{V_{r1s} = 14.1}$　kips

but $\leq$　$V_{r2s} := \phi_v \cdot \left(0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s \right)$

$\boxed{V_{r2s} = 24.5}$　kips

$$V_{rs} := \min\left(V_{r1s}, V_{r2s} \right)$$

$\boxed{V_{rs} = 14.1}$　kips

Check that the provided shear capacity is adequate:

Is $V_u = 12.2$ kip $\leq$ $V_{rs} = 14.1$ kip ?

$\boxed{check = \text{"OK"}}$

Note:　For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than: **LRFD[5.12.7.3]**

$0.0948 \cdot \lambda \sqrt{f'_c}$　$b \cdot d_s$

V_c for slabs simply supported need not be taken to be less than:

$0.0791 \cdot \lambda \sqrt{f'_c}$　$b \cdot d_s$

$\lambda = 1.0$　　(normal wgt. conc.) **LRFD [5.4.2.8]**

LRFD [5.7] and **LRFD [5.12.8.6]** apply to slabs of box culverts with less than 2.0 ft of fill.

Check that the nominal shear resistance, V_n, of the concrete in the walls is adequate for shear without shear reinforcement per **LRFD [5.7.3.3]**. Calculations shown are for the exterior wall.

$$V_n = V_c = 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v \leq 0.25 \cdot f'_c \cdot b_v \cdot d_v$$

$\beta := 2$　　　　　**LRFD [5.7.3.4.1]**

$f'_c = 3.5$　　　　culvert concrete strength, ksi

$b_v := 12$　　　　effective width, in

$h := t_{wex}$　　　　height of concrete design section, in　　　$h = 12.00$　in

$\lambda = 1.0$　　　　normal wgt. conc. **LRFD [5.4.2.8]**

Distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - \text{cover} - \frac{\text{Bar}_D(\text{Bar}_{No})}{2}$$

$\boxed{d_s = 9.69}$ in

The effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; **LRFD [5.7.2.8]**

$$d_{v\_i} = d_s - \frac{a}{2}$$

from earlier calculations:

$\boxed{\beta_1 = 0.85}$

$\boxed{f_s = 60}$ ksi

$\boxed{A_{s\_XW} = 0.40}$ in²

The distance between the neutral axis and the compression face:

$$c := \frac{A_{s\_XW} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_v}$$

$\boxed{\beta_1 = 0.85}$ $\boxed{\alpha_1 = 0.85}$ $\boxed{c = 0.79}$ in

$$a := \beta_1 \cdot c$$

$\boxed{a = 0.67}$ in

The effective shear depth:

$$d_{v\_i} := \left(d_s - \frac{a}{2}\right)$$

$\boxed{d_{v\_i} = 9.35}$

d_v need not be taken to be less than the greater of 0.9 d_s or 0.72h (in.)

$$d_v := \max\left(d_{v\_i}, \max\left(0.9 d_s, 0.72 t_{wex}\right)\right)$$

$0.9 \cdot d_s = 8.72$

$d_v = 9.35$ in

$0.72 \cdot t_{wex} = 8.64$

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_{r1w} := \phi_v \cdot \left(0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v\right)$$

$\boxed{V_{r1w} = 11}$ kips

but ≤ $$V_{r2w} := \phi_v \cdot \left(0.25 \cdot f'_c \cdot b_v \cdot d_v\right)$$

$\boxed{V_{r2w} = 83}$ kips

$$V_{rw} := \min\left(V_{r1w}, V_{r2w}\right)$$

$\boxed{V_{rw} = 11}$ kips

$$V_u := Vstr1_{XW}$$

$\boxed{V_u = 8.0}$ kips

Check that the provided shear capacity is adequate:

Is $V_u = 8.0$ kip ≤ $V_{rw} = 11.3$ kip ?

$\boxed{\text{check} = \text{"OK"}}$

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Table of Contents

39.1 General

39.1.1 Introduction

Signing is an integral part of the highway plan and as such is developed with the roadway and bridge design. Sign support structures are divided into two categories: Roadside signs, and Overhead Sign Structures (OSS). Roadside signs are designed and specified by the roadway engineer. OSS are designed by a Department (in-house or consultant) structural engineer or by a contractor, depending on the type of OSS.

Generally, an OSS is comprised of three components: the sign(s), the structure, and the foundation. Signage details are provided in the WisDOT Sign Plate Manual referenced below. This chapter of the WisDOT Bridge Manual (BM) governs the design of the structure and the foundation for OSS.

Regional traffic engineers determine the type of overhead sign structure that meets the signage needs for a particular project. Selection guidance and information is provided in the Facilities Development Manual (FDM) 11-55-20. That selection is communicated to the Bureau of Structures through the SSR submittal process.

The responsibility for developing contract plans depends on the type of sign structure selected and may be the role of Bureau of Structure staff, Regional staff, or engineering consultants.

39.1.2 Sign Structure Types and Definitions
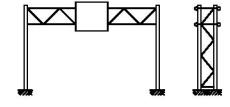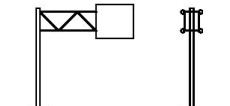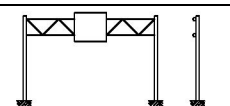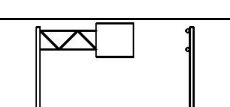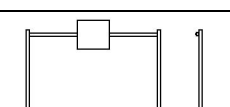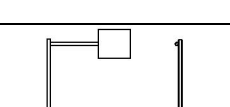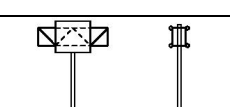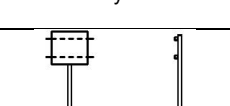
Roadside Sign: Refers to roadside signs supported on ground mounted posts adjacent to roadways. Ground mounted sign support posts are not considered "structures" and as such, are not assigned a structure number. See WisDOT Sign Plate Manual for details.

https://wisconsindot.gov/Pages/doing-bus/local-gov/traffic-ops/manuals-and-standards/signplate/signplate.aspx

Overhead Sign Structure (OSS): Refers to structural supports for mounting signs over a roadway. OSS are assigned a structure number and inventoried in WisDOT's Highway Structures Information (HSI) system. These structures are included on the section 8 structure sheets of a contract plan set.

In prior editions of the Bridge Manual there were two categories of overhead sign structures - "Sign Bridges" and "Overhead Sign Supports (OHSS)". Sign bridges were Department designed, and OHSS were contractor designed. While the roles of design remain the same, this edition shifts away from that terminology, instead focusing on terminology that best describes the geometric characteristics of the sign structure.

Table 39.1-1 summarize OSS types used by WisDOT:

Overhead Sign Structure Type	Description	Standard Structure Design	Standard Foundation Design
Full Span 4-Chord Truss	A 4-chord space truss with dual, trussed vertical support posts at each end. Used to support large Type I static highway sign panels and Dynamic Message Signs (DMS). Typically used over multi-lane state highways and interstate routes.	Yes	Yes
Cantilever 4-Chord Truss	A 4-Chord space truss with a single vertical support post. Used to support large Type I static highway sign panels and DMS. Commonly used to span over the outside lanes of multi-lane state highways and interstate routes to delineate exit lanes and ramps.	Yes	Yes
Full Span 2-Chord Truss	A 2-chord planar truss with single vertical support posts at each end. Used to support Type II and smaller Type I static signs and DMS over roadways and state highways.	No	Yes
Cantilever 2-Chord Truss	A 2-chord planar truss with a single vertical support post. Used to support Type II and smaller Type I static signs and DMS over roadways and state highways.	No	Yes
Full Span Monotube	Similar to a Full Span 2-Chord Truss but with only a single horizontal sign support member. Used to support small Type II static signs.	No	Yes
Cantilever Monotube	Similar to a Cantilever 2-Chord Truss but with only a single horizontal support member. Used to support small Type II static signs.	No	Yes
Butterfly Truss	A 4-Chord space truss with a centrally located single vertical support post used to support DMS. Typically used in the medians of multi-lane interstate routes.	No	No
Butterfly	Similar to a Butterfly Truss but with multiple monotube horizontal sign support members.	No	No
Bridge Mounted Sign Support	Sign support brackets to mount signs to the sides of grade separation highway bridges over the underpass roadway. These are typically used in special circumstances where other OSS types cannot be used.	No	NA
Note: Tri-chord and dual non-trussed monotubes are not permitted (except for Butterfly type)			

Table 39.1-1
WisDOT Overhead Sign Structure Types

39.1.3 Additional Terms

Type I Sign: Larger signs on an extruded aluminum base material, typically mounted on steel I-beams. Large guide and message signs with green backgrounds on interstate routes are Type I signs.

Type II Sign: Signs consisting of direct applied message on either plywood or sheet aluminum base material, typically mounted on wood or steel posts.

Dynamic Message Sign (DMS): An electronic traffic sign, often used in urban settings to inform drivers of unique and variable information. These signs are generally smaller in wind loaded area than Type I signs, but are heavier and load the truss eccentrically.

OSS Standard Designs: A group of pre-designed sign structures. The standard design includes both the structure and its foundation. The limitations for use is provided in section 39.1.5 and 39.1.6. See for further information on OSS Standard Designs.

OSS Non-Standard Design: Refers to sign structures that fall outside the OSS Standard Design parameters. It also applies to sign structure types not covered by standard design. These sign structures require a structural engineer provide a unique individual design of the structure and/or its foundation. See 39.4.5 for further information on OSS Non-standard Designs.

OSS Contractor Designed: Refer to sign structures that are designed and detailed by the contractor as part of the construction contract. The limitations for use is provided in section 39.1.5 and 39.1.6. The contractor does not design the foundation. For this, pre-designed foundations are available for use with these types of sign structures. See 39.4.6 for further information on OSS Contractor Designed.

OSS Standard Design Drawings: Refers to a library of WisDOT developed detail drawings for the *OSS Standard Designs* and the foundations for *OSS Contractor Designed*, otherwise indicated by a "yes" in Table 39.1-1. These standard design drawings are inserted into the contract plans with no additional design or detailing effort required.

39.1.4 OSS Selection Criteria

Chapter 11-55-20 of the Facilities Development Manual (FDM) provides selection guidance for determining sign structure type. The selection guidance was developed based on the design limitations of Table 39.1-1 and Table 39.1-2 and the information provided in the OSS Standard Design Drawings.

39.1.5 Cantilever OSS Selection Criteria

Cantilever OSS Type	Design	Cantilever Length [1]	Vertical Support Height [1]	Static Sign Total Area & Max. Dimensions [2]		DMS Total Area & Weight [1]
Monotube	Contractor Designed	40'-0" Max.	25'-0" Max. Column Base Plate to CL of Monotube Arm	Sign Area ≤ 75 SF Max. Sign Height ≤ 5'-0"		Not Used
2-Chord Truss	Contractor Designed	40'-0" Max. (static) / 20'-0" Max. (DMS)	27'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 150 SF Max. Sign Height ≤ 10'-0"		13'-9"W x 8'-0"H Max. 750 Lbs. Max
4-Chord Truss	Standard Design	20'-0" Min. 30'-0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 264 SF Max. Sign Height ≤ 15'-0"	OR	19'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Standard Design	>30'-0" 38'-0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 240 SF Max. Sign Height ≤ 15'-0"		19'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Non-Standard Design	>38'-0"	Column Height Exceeds Limit for Standard Design	Sign Area or Max. Sign Height Exceeds Limits For Standard Design		DMS Dimensions or Weight Exceeds Limits For Standard Design

Table 39.1-2
Cantilever 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: Static Type I sign panels may extend 1'-0" beyond end of Cantilever 4-Chord Truss.

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 $\leq$ 150 SF Max. Sign Height $\leq$ 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	150 SF < Sign Area $\leq$ 300 SF Max. Sign Height $\leq$ 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	300 SF < Sign Area $\leq$ Note 2 Max. Sign Height $\leq$ 12'-0"		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).

39.1.7 Butterfly and Butterfly Truss OSS

OSS Type	Design	Static Sign Total Area & Max. Dimensions[2]	OR	DMS Total Area & Weight
Butterfly	Non-Standard Design	Sign Area ≤ 240 Sq. Ft. Sign Height ≤ 10'-0"		N.A.
Butterfly Truss [1]	Non-Standard Design	Sign area > 240 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.

39.2 Materials

WisDOT uses the following for round, hollow structural sections (HSS) for truss chord members, vertical support members and horizontal monotube members.

Member Type	Material Requirements	
HSS Chords, Vertical Supports, & Horizontal Monotubes	Wall Thickness ≤ ½"	ASTM A500, Grade C (Fy = 46 ksi)
	Wall Thickness > ½" and Pipe Diameter ≤ 20"	ASTM A1085 (round HSS) Or API 5L Grade 46 PSL-2 (round pipe)
	Pipe Diameter > 20" (Any Wall Thickness)	API 5L Grade 46 PSL-2 (round pipe)
Plates, Bars, and Structural Angles	ASTM A709, Grade 36	
Round or Multi-Sided Tapered Poles	ASTM A595, Grade A (Fy = 55 ksi) Or ASTM A572, Grade 55	

Galvanized ASTM F3125 A325 bolts with DTI washers are to be used in all primary structural connections, including those that are fully tensioned. A449 bolts are not allowed in fully tensioned connections and are only allowed in full span chord to column saddle or full span post to chord clamp connections. More details can be found in the OSS Standard Design Drawings and Standard Specifications Section 532.

WisDOT policy item:

Installation of flat washers in between faying surfaces of mast arm connection plates is not allowed.

When selecting members sizes for individually designed OSS, it is important to select members that are regularly produced and domestically available. Specifying members that are infrequently produced may result in higher bid prices, longer fabrication lead time, and/or member substitution requests that may delay the fabrication and production process. A general rule of thumb is to select HSS round tube members that match standard (Schedule 40) outside pipe diameters and thickness. The Steel Tube Institute provides current information on their website regarding domestic availability of HSS sections at:

https://steeltubeinstitute.org/hss/availability-tool/.

Designers can also consult the Bureau of Structures.

39.3 Specifications

39.3.1 LRFD Design

WisDOT has transitioned the design of all roadside standard Type 1 breakaway sign supports and foundations to be in accordance with the *AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, 1st Edition (2015) (LRFDLTS-1)* with current Interim revisions.

WisDOT is currently transitioning the design of OSS to be in accordance with the AASHTO LRFDLTS-1 with current Interim revisions. Use of the AASHTO LRFDLTS-1 specification for OSS is currently optional and will be required beginning with the December 2020 letting.

39.3.2 Other Specifications and Manuals

The following manuals and specifications provide further guidance and requirements for the design and construction of OSS:

- Wisconsin Department of Transportation "*Bridge Manual*" (BM)

- Wisconsin Department of Transportation "*Geotechnical Manual*"

- Wisconsin Department of Transportation *"Facilities Development Manual"* (FDM)

- State of Wisconsin *"Standard Specifications for Highway and Structure Construction"*

- State of Wisconsin "*Construction and Materials Manual*" (CMM)

- AASHTO "*LRFD Bridge Design Specifications"* (Current Edition and Interim Specifications)

- American Society for Testing and Materials Standards (ASTM)

- American National Standards Institute / American Petroleum Institute 5L Specification for Line Pipe.   (ANSI / API 5L)

- AWS D1.1 Structural Welding Code (Steel)

- AWS D1.2 Structural Welding Code (Aluminum)

39.4 Design Considerations

39.4.1 Roadside Signs

Supports for roadside signs are of two types, depending upon the size and type of the sign to be supported. For small signs, the column supports are treated timber embedded in the ground. For larger Type 1 signs and DMS, the columns are galvanized steel supported on drilled shafts. Standard design and support estimates are given in the A3 Series of the "Sign Plate Manual."

Roadside sign supports are located behind existing or planned guardrail as far as practical from the roadway and out of the likely path of an errant vehicle. If roadside signs are located within the 30-foot corridor and not protected, break-away sign supports are detailed. Roadside sign supports for DMS, which includes dynamic message signs and variable message signs, are to be protected by concrete barrier or guardrail.

Currently, all steel column supports for roadside Type 1 signs, and DMS are designed to break-away upon impact.

The Wisconsin DOT Bureau of Traffic Operations has standard designs and details available for Type 1 Roadside Sign supports and foundations. The standard steel post design tables provide maximum sign mounting heights. If a sign configuration is required that does not fall within the limits of the standard designs, the sign support must be designed by a structural engineer. The design must be in compliance with the applicable specifications listed in 39.3. The Type 1 roadside sign standard foundation designs are based on the assumptions of cohesionless soils with the following properties:

- Soil Unit Weight = 115 pounds per cubic foot
- Angle of Internal Friction = 24 degrees
- Soil Modulus Parameter = 25 pounds per cubic inch

Wisconsin has standard design and details available for DMS roadside sign supports. If weaker subsurface conditions are known or suspected, a subsurface soil investigation per 39.5 would be implemented to gather necessary design information.

39.4.2 Overhead Sign Structures (OSS)

39.4.2.1 General

OSS types and names used by WisDOT are summarized in Table 39.1-1.

The connections of web members to chords are designed for bolted or shop welded connections to allow the contractor the option to either galvanize individual members or complete truss sections after fabrication. Steel base plates are used for anchor rod support attachment.

Aluminum sign structures are currently not being designed for new structures. Rehabilitation and repair type work may require use of aluminum members and shall be allowed in these

limited instances. The following guidelines apply to aluminum structures in the event of repair and rehabilitation type work.

Aluminum sign structure trusses are designed and fabricated from tubular shapes shop welded together in sections. The minimum thickness of truss chords is 1/4-inch and the minimum outside diameter is 4 inches. The recommended minimum ratios of "d/D" between the outside diameters "d" of the web members and "D" of chord members is 0.4. A cast aluminum base plate is required to connect the aluminum columns to the anchor rods. AASHTO Specifications require damping or energy absorbing devices on aluminum overhead sign support structures to prevent vibrations from causing fatigue failures. Damping devices are required before and after the sign panels are erected on all aluminum sign bridges. Stock-bridge type dampers are recommended.

39.4.2.2 Vehicular Protection

Vertical supports for OSS Standard Designs are not designed for vehicular impact loads and must meet clear zone or barrier protection requirements in the FDM. Generally, all overhead sign structure vertical supports are located at the edge of shoulder adjacent to the traveled roadway and placed behind roadside concrete barriers or barrier type guardrail. See the FDM 11-55-20.6 for information on shielding requirements. Sign supports protected by roadside barriers or guardrail with adequate barrier deflection clearance between the backside of the barrier and the sign support are not required to be designed for Extreme Limit State vehicular collision loads.

When protection is not feasible, the vertical supports shall be designed with applicable Extreme Event collision loads in accordance to 13.4.10. This typically requires the use of a special, individually designed reinforced column and foundation to resist the large vehicular impact loads. In this situation the sign structure would be a non-standard design and BOS or an engineering consultant would need to provide the design.

39.4.2.3 Vertical Clearance

As provided in the FDM 11-35-1 Attachment 1.8, a minimum vertical clearance of 18'-3" is required for most routes. For sign structures over a designated High Clearance Route, 20'-3" above the roadway is required. See FDM 11-35-1 Attachment 1.9 for clearances relating to existing sign structures.

39.4.2.4 Lighting and DMS Inspection Catwalks

Lighting is no longer required on sign structures. Catwalks are only on 4-chord cantilever and full span OSS with DMS. When catwalks are provided for OSS supporting a DMS, additional vertical height must be provided to meet the vertical clearance requirements in 39.4.2.3 to the bottom of the catwalk brackets. Catwalk grating and toe plates shall be galvanized steel.

Along with inspection catwalks, all DMS OSS require hand holes, rodent screens and electrical conduits through the foundation to one of the vertical support posts to route electric power to the DMS. Standard Details are provided on the BOS website.

39.4.2.5 Signs Mounted on the Side of Grade Separation Bridges

When no practical alternatives exist, signs may be mounted on the side of grade separation bridges. This application requires individually designed structural mounting brackets to attach the sign to the side of the grade separation bridge. Wisconsin allows sign attachments orientated up to a maximum of a 20-degree skew to the roadway below. Any grade separation bridge structure with greater skew requires the mounting brackets to attach signs so they are orientated perpendicular to the roadway below.

Where possible, the depth of bridge mounted signs should be limited so the top of the sign does not extend above the top of the bridge parapets or railing. Signs are not permitted to extend below the bottom of the bridge girders. The minimum vertical clearance is set slightly above the clearance line of the overpass structure for signs attached directly to the side of a bridge.

Signs attached to the side of bridge superstructures and retaining walls are difficult to inspect and maintain due to lack of access. Attachment accessories are susceptible to deterioration from de-icing chemicals, debris collection and moisture. Therefore, the following guidance should be considered when detailing structure mounted signs and related connections:

1. Limit the sign depth to a dimension equal to the bridge superstructure depth (including parapet) minus 3 inches.

2. Provide at least two support connections per bracket.

3. Utilize cast-in-place anchor assemblies to attach sign supports onto new bridges and retaining walls.

4. Galvanized or stainless-steel adhesive concrete masonry anchor may be used to attach new signs to the side of an existing grade separation bridge or retaining wall orientated for shear load application only. Overhead anchor installation (direct pullout loading on anchor) is not allowed. Reference 40.16 for applicable concrete masonry anchor requirements.

39.4.2.6 Sign Structures Mounted on Bridge Pedestals

This refers to sign structures mounted across the top of roadways carried by a bridge structure. Sign structures can be mounted directly to the top of pier caps. This requires the pier cap to be extended beyond the limits of the superstructure width. Sign structures mounted to pier caps are not affected by superstructure deflections. Wisconsin allows sign attachments oriented up to a maximum of a 20-degree skew to the roadway below. Any grade separation bridge structure with a greater skew requires the mounting brackets to attach signs so they are oriented perpendicular to the roadway below.

Span live load deflections of the vehicular bridge superstructure affect sign structures mounted on to bridge superstructure concrete barrier pedestals. The magnitude of sign structure deflections and duration of sign structure vibrations is dependent on the stiffness of the girder and deck superstructure, the location of the sign structure on the bridge, and

the ability of the sign structure to dampen those vibrations out; among others. These vibrations are not easily accounted for in design and are quite variable in nature. For these reasons, the practice of locating sign structures on highway bridges should be avoided whenever possible.

The following general guidance is given for those instances where locating a sign structure on a bridge structure is unavoidable. This may occur due to the length of the bridge, or a safety need to guide the traveling public to upcoming ramp exits or into specific lanes on the bridge.

1. Locate the sign structure pedestals at pier locations.

2. Build the sign structure base off the top of the pier cap.

3. Provide adequate set back of the tower support of the sign structure behind the face of the parapet to avoid snagging of vehicles making contact with the parapet. See FDM 11-45-2.3.6.2.3 for information on required set back distances.

4. Use single pole sign supports (equal balanced butterfly's) in lieu of cantilevered (with an arm on only one side of the vertical support) sign supports.

5. Consider the use of a Stockbridge type damper in the horizontal truss of these structures.

6. Do not straddle the pier leaving one support on the pier and one support off the pier in the case of skewed substructure units for full span sign bridges

39.4.3 LRFD Requirements and WisDOT Guidance for OSS Design

39.4.3.1 Loads, Load Combinations, and Limit States

All OSS are to be designed per the AASHTO LRFDLTS-1. The following LRFD specification requirements are highlighted:

Design Wind Speed Recurrence Interval:

- Full Span 4-Chord Truss Sign Structures are designed for a basic wind speed recurrence interval of 1,700 years as defined in the AASHTO LRFDLTS-1 Specifications.

- All other OSS shall be designed for a basic wind speed recurrence interval of 700 years as defined in the AASHTO LRFDLTS-1 Specifications.

Wind load and wind load combinations shall be applied and investigated per AASHTO LRFDLTS-1. In general, horizontal wind pressure is applied normal to the center of gravity of exposed horizontal members and sign panels. For the design of vertical supports, three wind load cases are investigated and applied to the entire structure to determine the controlling wind load effect on the vertical supports:

Wind Load Case	Description	Normal Wind Component	Transverse Wind Component
1	Full Wind Normal to the Plane of the Structure	100%	0%
2	Full Wind Transverse to the Plane of the Structure	0%	100%
3	75% Full Wind in Both Directions Simultaneously	75%	75%

Figure 39.4-1
AASHTO LRFDLTS-1 Vertical Support Load Cases

Design sign area assumed for standard designs accommodates 12-foot high sign panels over 90% of the span length for full span 4-chord truss OSS. In the case of a proposed non-standard OSS with a required span length of 130 feet or less, the non-standard OSS should be individually designed for the actual anticipated sign panel area or DMS and mounting locations. In the case of a proposed non-standard OSS with required span length greater than 130 feet, the Bureau of Structures should be consulted to confirm the design sign area to be used for the design of the non-standard OSS.

Applied design wind pressure is determined for individual truss members and sign panels per the AASHTO LRFDLTS-1 specification Section 3.8. WisDOT design practice is to assume members located directly behind sign or DMS panels are shielded from wind exposure and are not loaded with wind pressure. No wind shielding is assumed provided to members that directly align with each other in plan or elevation views, but are several feet apart. This means no shielding effect is assumed for members in the front and back truss planes of a 4-chord truss even if the members are perfectly aligned. For example, viewing a 4-chord truss in elevation view, members in the front truss plane, located directly behind a sign panel would assumed to be shielded from wind pressure by the sign panel, but members in the back-truss plane would assumed to be loaded with wind pressure, despite also being behind the sign panel or aligned with other members in the front truss plane.

Strength 1 load combinations in AASHTO LRFDLTS-1 include only dead load and live load. A 500-pound live load distributed over 2'-0" transversely to the member, only applies to catwalks and catwalk support brackets when catwalks are included for OSS with DMS. The Strength 1 load combination may control the design of the catwalk and catwalk support brackets, but does not control the design of the truss superstructure the catwalk brackets are attached to. For OSS carrying static Type 1 signs, the Strength I load combination includes only dead load and does not control.

Load combinations that include wind generally control the design of sign structures. A change in the AASHTO LRFDLTS-1 specification is that load combinations that include wind are considered Extreme Event load cases.

AASHTO LRFDLTS-1 specifications do not define an ice loading and leave it to the discretion of individual owners to consider and specify an ice loading if warranted in their climate. WisDOT policy is to maintain consideration of an ice load and include in the Extreme Event I load combination.

Load Combinations are as follows:

Strength I: 1.25 DL + 1.6 LL

Extreme Event I (Load Case 1):   1.1 DL + 1.0 ICE + W (Max. DL and ICE effects)

Extreme Event I (Load Case 2):   0.9 DL + W (Min. DL and no ICE effects)

Ice build-up is modeled as a 3 psf load applied to the exposed surface area (circumference) of truss members. It is not necessary to increase the wind pressure load on truss members due to increased member exposure area caused by ice build-up. Ice load is applied to only the front face of sign or DMS panels.

For vertical column support members, W in the above Extreme Event load cases is the controlling wind load case as specified in Figure 39.4-1.

39.4.3.2 Serviceability

Serviceability checks should conform to 10.4 and 10.5 of AASHTO LRFDLTS-1. However, the vertical deflection limit L/150 shall include ICE load, applied per 39.4.3.1.

39.4.3.3 Fatigue

AASHTO LRFDLTS-1 specifies three fatigue loads to check against member and connection fatigue stress range limits as follows:

Galloping – AASHTO LRFDLTS 11.7.1.1:   Applies to all cantilever OSS,
                                                                        except cantilever 4-chord truss

Natural Wind Gust – AASHTO LRFDLTS 11.7.1.2:   Applies to all OSS.

Truck Induced Gust – AASHTO LRFDLTS 11.7.1.3:   Applies to all OSS.

Truck induced gust pressure is applied in the upward direction and reduces with increasing height. Truck induced gust pressure applied to truss members in the top horizontal truss plane, will be less than truck induced gust pressures applied to truss members in the bottom horizontal truss plane. Since truck induced gust pressure is acting upward, Type 1 static signs receive and transmit only minimal gust pressure due to their narrow profile when viewed in plan. DMS however, have considerable width and "wind exposure area" when viewed in plan. Truck induced gust pressure can impart a significant upward pressure on DMS that also creates a torque on the truss superstructure due to the offset between the center of gravity of the DMS and the truss superstructure.

39.4.3.4 Connection Design

WisDOT policy item:

Bureau of Structures policy is to design welded and bolted connections per the applicable provisions of the current AASHTO LRFD Bridge Design Specifications. This is a deviation from the AASHTO LRFDLTS-1, which refer the design of welded connections to the AWS D1.1 Structural Welding Code.

For truss superstructures, current practice is to design and provide alternate details of the connection of web members (angles) to main chord members (HSS tubular round sections) for both welded and bolted connections, except the chord to column connection and first panel of cantilever trusses which must be bolted. This affords the fabricator the option of galvanizing individual members prior to truss fabrication (using bolted connections) or galvanizing entire truss segments after assembly (using bolted or welded connections).

39.4.4 OSS Standard Designs

Standard Design OSS types and limitations are listed in Table 39.1-2 and Table 39.1-3. Because these structures are pre-designed and pre-detailed the involvement of a Department structural engineer is usually not required. Bureau of Structures is responsible for maintaining and updating the Standard Designs as needed.

The Standard Design OSS types were developed to cover a wide range of signage requirements while placed over typical roadway and roadside configurations. Standard Designs are not intended to cover unique situations or unusual geometry, or for reasons described in 39.4.5. Contact the Bureau of Structures Design Section with questions regarding applicability of standard designs.

Standard Foundation Designs are included in the OSS Standard Detail Drawings. See 39.5 for further guidance on subsurface investigation, assumed soil parameters, and foundation design.

When Standard Design OSS are used in a project, the Region or their consultant prepare final contract plans as described in 39.1.9. This includes filling out the appropriate sign structure specific information on both the General Notes and General Layout sheets and combining those with the appropriate OSS Standard Design Drawings to make up the final plan set.

39.4.5 OSS Non-Standard Designs

Design and plan detailing must be provided by Bureau of Structures or by a structural design consultant for all non-standard designs. The following circumstances warrant a non-standard design:

1. The OSS type is Butterfly, Butterfly Truss, or Bridge Mounted

2. The OSS type falls outside the limits of span length, sign area, DMS weight, or sign height in FDM 11-55-20 Figure 20.2.3 and Figure 20.2.4.

3. Region soil engineer advises that subsurface conditions at the site are expected to negatively differ from assumed soil profile and design parameters of standard foundations (e.g. soft soil or shallow bedrock – see 39.5.2.2).

4. Excessive sign structure height (e.g. sign structure behind MSE wall) or requires the use of concrete column (designed for impact load – see 39.4.2.2)

BOS must be consulted to verify and confirm the need for individual designs before undertaking this effort.

The design detailing shall generally follow the guidance provided by the OSS Standard Design Drawings but should clearly delineate any required changes to individual member sizes, connections and foundation details necessary to satisfy the AASHTO LRFDLTS-1 Design Specifications.

In some instances, it may still be appropriate to use part or all of the Standard Designs even though the sign structure is considered a Non-standard Design. A couple of examples include:

1. A sign structure has both static and DMS sign types specified for mounting (consult with BOS before using a standard design in this situation).

2. A Standard Design structure is used in conjunction with a Non-standard foundation. See section 39.5.3.

In any case, the sign structure is still considered a Non-standard design in terms of the design process and should proceed as detailed in 39.1.9.

39.4.6 OSS Contractor Designed

Contractor Designed OSS types and limitations are listed in Table 39.1-2 and Table 39.1-3. Because these structures are designed by the contractor, involvement of a Department structural engineer is usually not required.

Standard Foundation Designs are included in the OSS Standard Detail Drawings. See 39.5 for further guidance on subsurface investigation, assumed soil parameters, and foundation design. Bureau of Structures is responsible for maintaining and updating the standard foundation designs that go along with the Contractor Designed OSS types.

These structures are designed for the required actual sign area and configuration, unless future expansion is anticipated, which should be noted and shown on the plans. The required actual sign area, span length, etc. is used to select the appropriate standard foundation from the figure provided in chapter 11-55-20 of the FDM.

When Contractor Designed OSS are used in a project, the Region or their consultant prepare final contract plans as described in 39.1.9. This includes filling out the appropriate sign structure specific information on both the General Notes and General Layout sheets and combining those with the appropriate OSS Standard Design Drawings to make up the final plan set.

39.5 Geotechnical Guidelines

39.5.1 General

For full span and cantilever 4-chord trusses, the typical preferred foundation is comprised of two cylindrical drilled shafts connected by a concrete cross-girder, as detailed in the OSS Standard Design Drawings. The top of the cross-girder is set 3 feet above the highest ground elevation at the foundation. For all other types, the typical preferred foundation is comprised of a single cylindrical drilled shaft directly supporting the column vertical support. Occasionally, some columns are mounted directly on top of modified bridge parapets, pier caps and concrete towers instead of footings.

There are several potential challenges regarding subsurface exploration for OSS foundations:

- The development and location of these structures are typically not known at the onset of the preliminary design stage, when the most subsurface exploration typically occurs. This creates the potential need for multiple drilling mobilizations for the project.

- OSS are often located in areas of proposed fill soils. The source and characteristics of fill soil is unknown at the time of design.

- OSS foundations are often located on the shoulder or median directly adjacent to high-volume roadways. Obtaining boings in these locations typically requires significant traffic control, night work, and working in a potentially hazardous work zone.

- If a consultant is involved in the project, the unknowns associated with these structures in the project scoping stage complicate the consultant contracting process. It is often difficult to determine the need for OSS specific subsurface investigation at the time the consultant contract is normally being scoped. In cases where the need for a specific subsurface investigation is known or anticipated, an assumption must be made regarding the level of subsurface investigation to include in the consultant design contract. Alternatively, a decision can be made to assume use of standard OSS and foundation designs. If the need for specific subsurface investigation is later determined to be necessary, this may require a change to add it to the consultant contract.

39.5.2 Standard Foundations for OSS

39.5.2.1 General

WisDOT has created standard full span and cantilever 4-chord truss designs that include fully designed and detailed drilled shaft foundations as part of the overall standard design. The standard foundation details are incorporated with the OSS Standard Design Drawings for these structures and are available on the BOS website.

Single drilled shaft OSS Standard Design Drawings for use with contractor designed full span and cantilever 2-chord truss and monotube OSS are also available on the BOS website.

WisDOT has no standard foundation design details for alternate foundation types and the selected alternative foundations would be required to be individually designed and reviewed by BOS.

39.5.2.2 Design Parameters Used for Standard Foundation Design

Standard dual and single drilled shaft foundation designs were developed in accordance with applicable requirements of Section 10 of the *AASHTO LRFD Bridge Design Specifications.*

The standard foundation designs are based on the following design parameters:

- Total Unit Weight = 125 pcf

- Granular Soil Profile: Internal Angle of Friction = 24 degrees, or

- Cohesive Soil Profile: Undrained Shear Strength = 750 psf

- Soil and drilled shaft downward resistance factor ϕ =1.0 [1]

- Drilled shaft uplift resistance factor ϕ =0.8 [1]

- Depth of water table assumed 10 feet below the ground surface

- Soil side resistance is considered fully effective to the top of the drilled shaft or top of ground surface, whichever is the lower elevation.

- Lateral deflection at the top of the foundation limited to 1-inch at the Service I Limit State

   Note 1: Resistance factors per AASHTO 10.5.3.3 assuming the drilled shaft design is governed by the wind load combination which is an Extreme Event load combination.

WisDOT policy item:

Design of standard sign structure foundations assumes soil side resistance is fully effective to the top of the drilled shafts for full span 4-chord OSS foundations and to within 3 feet below the lowest ground surface for all other OSS foundations. This is a deviation from AASHTO 10.8.3.5 1b.

Use of the standard foundations requires that the in-situ soils parameters at the site meet or exceed the assumed soil design parameters noted above. Soil parameters were selected to be sufficiently conservative to cover most sites across the state**.** Designers should contact the Region Soils Engineer or the Geotechnical Consultant to assist in the evaluation of the subsurface conditions compared to the assumed soil parameters. An assessment can also be made by checking nearby borings and as-built drawings of nearby existing structures, and similar sources. If there is reason to suspect weaker soils or that shallow bedrock is present, OSS specific soil borings should be obtained to confirm in-situ soil properties meet or exceed the assumed parameters used for the standard designs. If these site-specific soil properties

do not meet the above minimums, a special individual foundation design will be required using actual soil parameters determined from a subsurface investigation per 39.5.3.

39.5.3 Standard Base Reactions for Non-Standard Foundation Design

There may be instances when a Standard Design sign structure is used in conjunction with a non-standard foundation, for reasons detailed in 39.4.5. Contact Bureau of Structures to obtaining the Standard Design or Contractor Designed sign structure base reactions that were used in developing the standard foundations.

39.5.4 Subsurface Investigation and Information

No subsurface investigation/information is necessary for the use of WisDOT standard foundations. Appropriate subsurface information is necessary for any non-standard OSS or situation that is outside any of the standard design ranges of applicability which requires an individual foundation design to be performed.

There may be several methods to obtain the necessary subsurface soil properties for a custom, individual foundation design, as described below:

- In areas of fill soils, the borrow material is usually unknown. The designer should use their best judgment as to what the imported soils will be. Standard compaction of this material can be assumed.

- The designer may have a thorough knowledge of the general soil conditions and properties at the site and can reasonably estimate soil design parameters.

- The designer may be able to use information from nearby borings. Judgment is needed to determine if the conditions present in an adjacent boring(s) are representative of those of the site in question.

- If the designer cannot reasonably characterize the subsurface conditions by the above methods, a soil boring and Geotechnical report (Site Investigation Report) should be completed. Necessary soil design information includes soil unit weights, cohesions, phi-angles and location of water table.

Designers, both internal and consultant, should also be aware of the potential of high bedrock, rock fills and the possible conflict with utilities and utility trenches. Conservative subsurface design parameters are encouraged.

39.6 Appendix – OSS Limiting Parameters
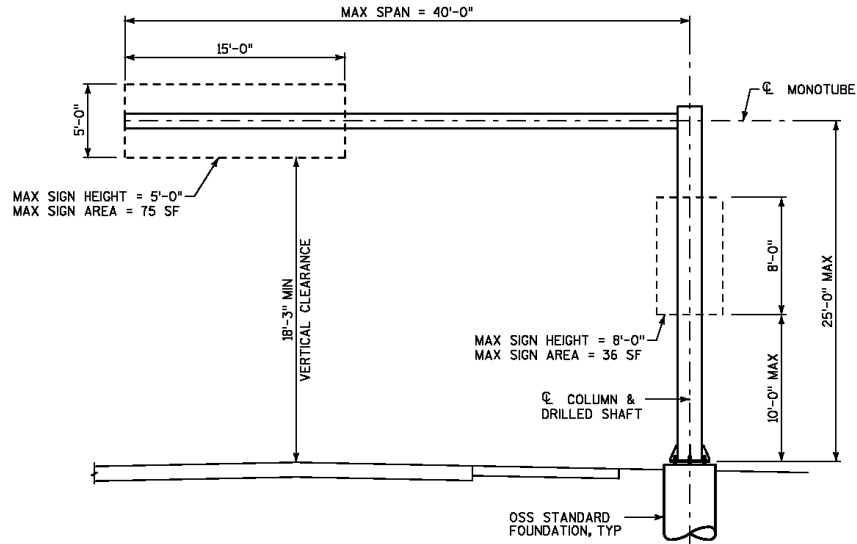
CANTILEVER MONOTUBE
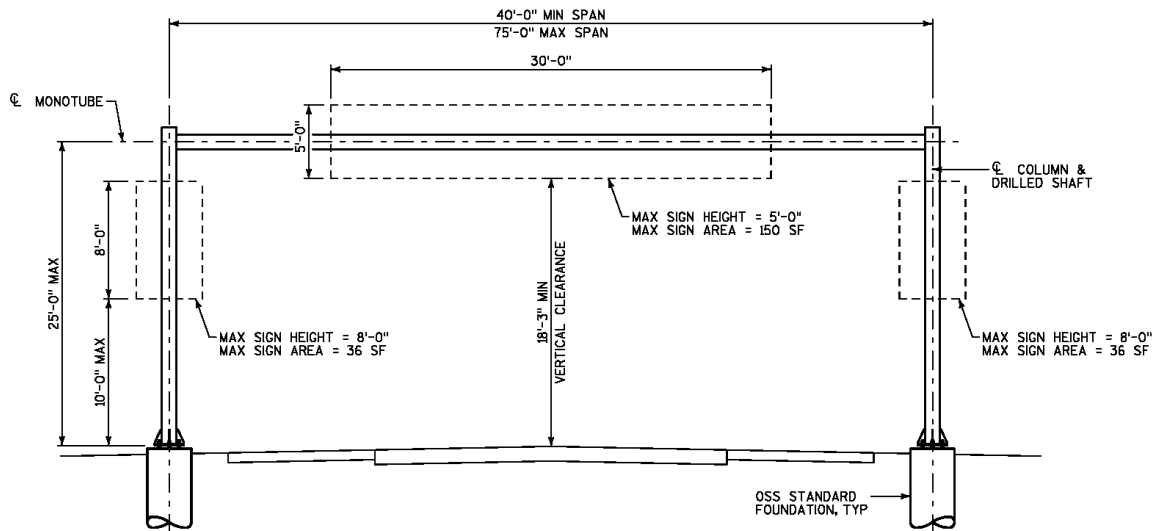
FULL SPAN MONOTUBE

DESIGNER NOTES:

1. SELECT STRUCTURE TYPE BASED ON REQUIRED SPAN LENGTH AND DESIGN SIGN AREA TO BE SUPPORTED ON THE STRUCTURE. SHOW ON THE "GENERAL LAYOUT" SHEET FOR THE STRUCTURE.

2. SELECT OSS STANDARD FOUNDATION TYPE AND SHOW ON THE "GENERAL LAYOUT" SHEET. SEE "OSS MONOTUBE & 2-CHORD TRUSS STANDARD FOUNDATIONS" SHEET AND SECTION 11-55-20 OF THE WISDOT FACILITIES DEVELOPMENT MANUAL FOR FOUNDATION SELECTION CRITERIA.

3. COLUMN SIGN AREA DOES NOT CONTRIBUTE TO THE SELECTION LIMITS OF THE FDM, BUT ARE PERMITTED UP TO THE LIMITS SHOWN IN THE FIGURES.
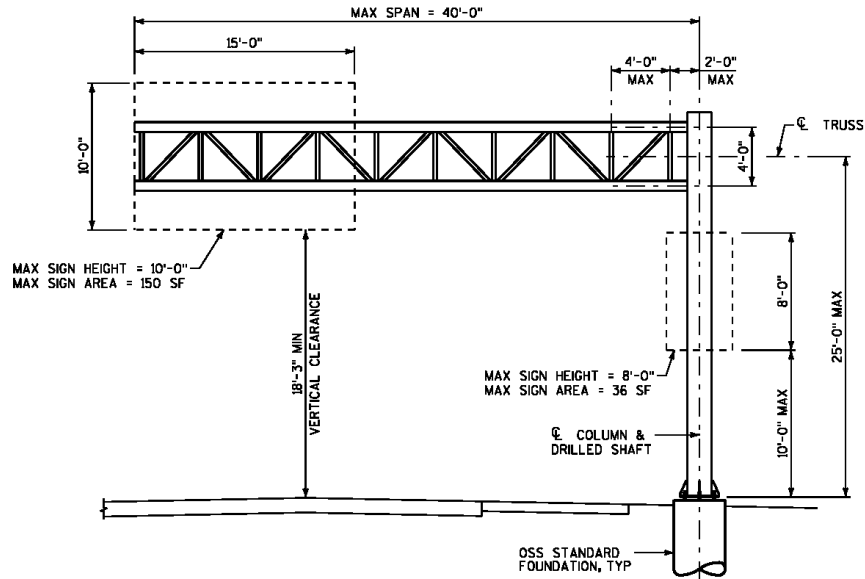
CANTILEVER 2-CHORD TRUSS

FULL SPAN 2-CHORD TRUSS

DESIGNER NOTES:

1. SELECT STRUCTURE TYPE BASED ON REQUIRED SPAN LENGTH AND DESIGN SIGN AREA TO BE SUPPORTED ON THE STRUCTURE. SHOW ON THE "GENERAL LAYOUT" SHEET FOR THE STRUCTURE.

2. SELECT OSS STANDARD FOUNDATION TYPE AND SHOW ON THE "GENERAL LAYOUT" SHEET. SEE "OSS MONOTUBE & 2-CHORD TRUSS STANDARD FOUNDATIONS" SHEET AND SECTION 11-55-20 OF THE WISDOT FACILITIES DEVELOPMENT MANUAL FOR FOUNDATION SELECTION CRITERIA.

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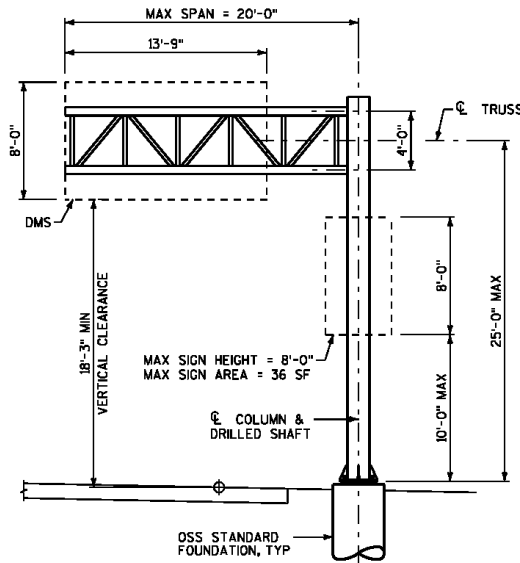
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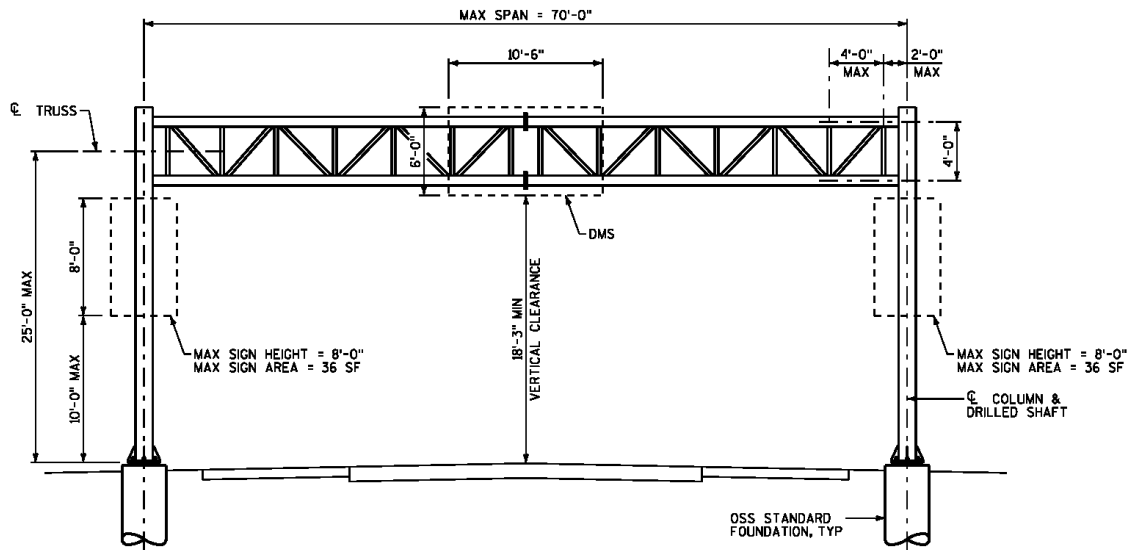
FULL SPAN 2-CHORD TRUSS DMS
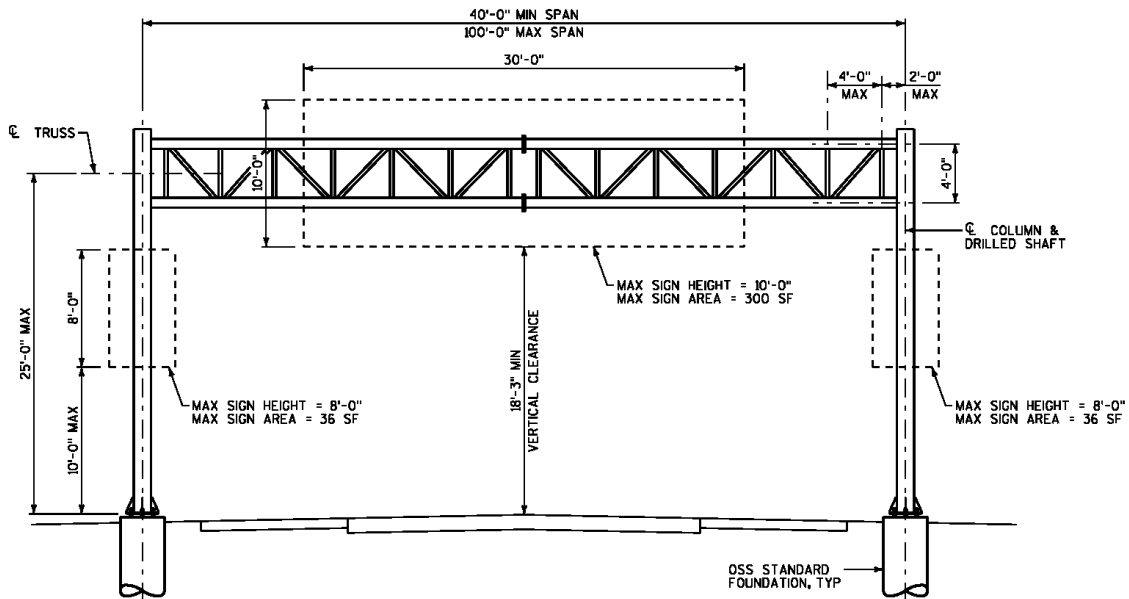
DESIGNER NOTES:

1. SELECT STRUCTURE TYPE BASED ON REQUIRED SPAN LENGTH AND DESIGN SIGN AREA TO BE SUPPORTED ON THE STRUCTURE. SHOW ON THE "GENERAL LAYOUT" SHEET FOR THE STRUCTURE.

2. SELECT OSS STANDARD FOUNDATION TYPE AND SHOW ON THE "GENERAL LAYOUT" SHEET. SEE "OSS MONOTUBE & 2-CHORD TRUSS STANDARD FOUNDATIONS" SHEET AND SECTION 11-55-20 OF THE WISDOT FACILITIES DEVELOPMENT MANUAL FOR FOUNDATION SELECTION CRITERIA.

3. COLUMN SIGN AREA DOES NOT CONTRIBUTE TO THE SELECTION LIMITS OF THE FDM, BUT ARE PERMITTED UP TO THE LIMITS SHOWN IN THE FIGURES.

39.7 Design Examples

E39-1   Design of Foundation Cap Beam / Integral Barrier TL-5 Loading

E39-2   Design of Sign Bridge Concrete Column for Vehicle Impact

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Table of Contents

40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

40.2 History

40.2.1 Concrete

Prior to 1975, all concrete structures were designed by the Service Load Method. The allowable design stress was 1400 psi based on an ultimate strength of 3500 psi except for deck slabs. In 1965, the allowable stress for deck slabs on stringers was reduced from 1400 to 1200 psi. The reason for the change was to obtain thicker concrete decks by using lower strength. The thicker deck was assumed to have more resistance to cracking and greater durability. During this time no changes were made in the physical properties of the concrete or the mixing quantities until 1974 when more cement was added to the mix for concrete used in bridge decks.

In 1980, the use of set retarding admixtures was required when the atmospheric temperature at the time of placing the concrete is 70°F or above; or when the atmospheric temperature is 50°F or above and it is expected that the time to place the concrete of any span or pour will require four or more hours. Retarding admixtures reduce concrete shrinkage cracking and surface spalling. Also, during the early 1980's a water reducing admixture was provided for Grades A and E concrete mixes to facilitate workability and placement.

40.2.2 Steel

Prior to 1975, Grade 40 bar steel was used in the design of reinforced concrete structures. This was used even though Grade 60 bars may have been furnished on the job. Allowable design stress was 20 ksi using the Service Load Method and 40 ksi using the Load Factor Method.

40.2.3 General

In 1978, Wisconsin Department of Transportation discovered major cracking on a two-girder, fracture critical structure, just four years after it was constructed. In 1980, on the same structure, major cracking was discovered in the tie girder flange of the tied arch span.

This is one example of the type of failures that transportation departments discovered on welded structures in the 1970's and '80's. The failures from welded details and pinned connections led to much stricter standards for present day designs.

All areas were affected: Design with identification of fatigue prone details and classifications of fatigue categories (AASHTO); Material requirements with emphasis on toughness and weldability, increased welding and fabrication standards with licensure of fabrication shops to minimum quality standards including personnel; and an increased effort on inspection of existing bridges, where critical details were overlooked or missed in the past.

Wisconsin Department of Transportation started an in-depth inspection program in 1985 and made it a full time program in 1987. This program included extensive non-destructive testing. Ultrasonic inspection has played a major role in this type of inspection. All fracture critical

structures, pin and hanger systems, and pinned connections are inspected on a 72-month cycle.

40.2.4 Funding Eligibility and Asset Management

Nationally, MAP-21 (2012) and the FAST Act (2015) have moved structures asset management to a more data-driven approach. Funding restrictions with regards to Sufficiency Rating, Structural Deficiency, and Functional Obsolescence have been removed or significantly revised. In place of these past restrictions, MAP-21 requires the development and approval of a statewide Transportation Asset Management Plan (TAMP). A key part of the WisDOT TAMP is the Wisconsin Structures Asset Management System (WiSAMS).

WiSAMS is being developed as a planning tool, which analyzes current structure inspection data, projects future deteriorated structure condition, and applies Chapter 42-Bridge Preservation to recommend appropriate structure work actions at the optimal time. WiSAMS is a tool for regional and statewide programming, and is not designed as an in-depth scoping tool. WiSAMS may provide an estimate of the appropriate work action, but an in-depth evaluation of the actual structure condition and appropriate scope of work (SSR) and consideration of other non-structural project factors (e.g. cost and functionality) is still required.

In Wisconsin, the Local Bridge Program, through State Statute 84.18 and Administrative Rule Trans 213, is still tied to historic FHWA classifications of Sufficiency Rating, Structural Deficiency, and Functional Obsolescence.

40.3 Bridge Replacements

Bridge preservation and rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. Ideal bridge preservation strategy is explained in Chapter 42-Bridge Preservation. This guide should be followed as closely as possible, considering estimated project costs and funding constraints.

See Faculties Design Manual (FDM) 11-40-1.5 for policies regarding necessary bridge width* and structural capacity.

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of ensuring some level of acceptable serviceability; however, structure preservation as explained in Chapter 42-Bridge Preservation should be followed as closely as possible, considering estimated project costs and funding constraints.

The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are adequate to safely carry present and projected traffic. Information which is helpful in determining structure adequacy includes structure inspection history, inventory data, traffic projections, maintenance history, capacity and route designations. The methods of rehabilitation are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to ensure that rehabilitation will remove all structural deficiencies. Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation is required. See FDM 11-40-1.5 for policies regarding bridge rehabilitation.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic. Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and riding surface. Wisconsin has experienced some cracking in the concrete overlays and parapets during rehabilitation construction. The apparent cause of cracking is related to traffic impact due to rough structure approaches. Particular attention should be given to providing a smooth transition on temporary approaches and over expansion joints on all bridge decks during rehabilitation. Other causes of cracking are related to shrinkage, temperature, wind induced vibrations, concrete quality and improper curing.

Options for deck rehabilitation are as follows:

1. Asphalt Patch

2. Asphalt or Polymer Modified Asphaltic Overlay

3. Concrete or Modified Concrete Patch

4. Waterproof Membrane with an Asphalt Overlay (currently not used)

5. Concrete Overlay - Grade E Low Slump Concrete, or Micro-Silica Modified Concrete

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project

    a. Deck condition equal 4 or 5 and;

    b. Wear course or wear surface less than or equal to 3.

    c. No roadway work scheduled for at least 3 years.

2. Interstate Bridge with Roadway Work

    a. Deck Condition less than or equal 4.

    b. Wear course or wear surface less than or equal to 4.

3. Rehab not needed on Interstate Bridges if:

    a. Deck condition greater than 4.

    b. Wear surface or wear course greater than or equal 4.

4. All Bridges

WisDOT policy item:

On major rehab work, build to current standards such as safety parapets, full shoulder widths, etc. Use the current Bridge Manual standards and tables. Exceptions to this policy require approval from the Bureau of Structures Development Section.

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      a. Evaluate cost of repeated maintenance, traffic control as well as bridge work when determining life-cycle costs.

      b. Place overlays on all concrete superstructure bridges if eligible.

      c. For all deck replacement work the railing shall be built to current standards.

5. All Bridges with Roadway Work

    Coordinate with the Region the required staging of bridge related work.

A number of specific guidelines are defined in subsequent sections. As with any engineering project, the engineer is allowed to use discretion in determining the applicability of these guidelines.

40.5 Deck Overlays

As a bridge deck ages, preservation and rehabilitation techniques are necessary to maximize the life of the deck and ensure a level of acceptable serviceability. Overlays can be a useful tool to extend the service life of structures. This section discusses several overlay methods, considerations, and guidelines for deck overlays. The provided information is intended for deck-girder structures and may be applicable for slab structures. Slab structures may have different condition triggers and may warrant additional considerations.

The following criteria should be met when determining if an overlay should be used:

- The structure is capable of carrying the overlay dead load

- The deck and superstructure are structurally sound

- The desired service life can be achieved with the considered overlay and existing structure

- The selected option is cost effective based on the anticipated structure life and funding constraints

Decks deteriorate at different rates depending on many factors, including deck materials, material quality, construction quality, structure geometry, exposure to deicing agents, and traffic demands. Additionally, there is a wide variance in the amount of structure preservation techniques utilized by different regions. While the deck age can be a useful parameter, it should not be the primary consideration for determining the eligibility of overlays. Recommended preservation techniques should rely heavily on quality inspection data to determine the appropriate course of action. For more information related to preservation techniques and practices, refer to Chapter 42-Bridge Preservation.

Overlays can be an effective tool to maximize the life of the deck. Figure 40.5-1 illustrates a possible preservation scenario using deck deterioration curves showing approximate deck NBI ratings at which the overlays would occur, and the benefit of performing these overlays. This scenario assumes that the underside of deck deterioration is significantly reduced due to the preservation techniques performed on the top side of the deck.
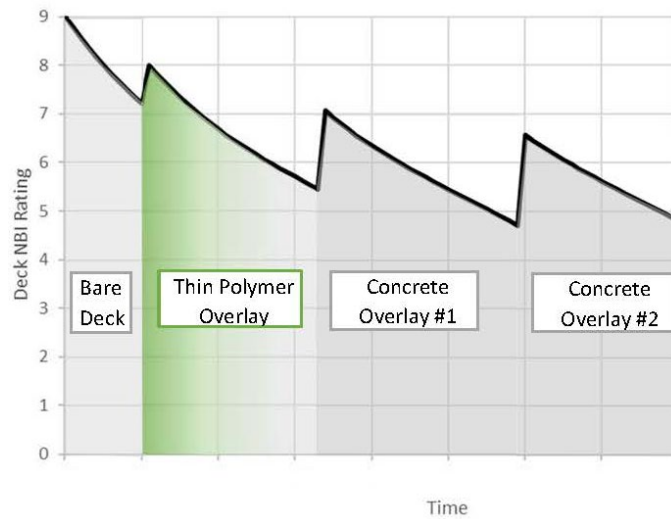
Figure 40.5-1
Deck Deterioration Curve

40.5.1 Overlay Methods

There are several commonly used overlay methods for the preservation and rehabilitation of decks. Generally, thin polymers overlays are recommended as preventative maintenance for decks with a minimal amount of deck distress. Ideally, thin polymer overlays are applied within the first couple of years to limit chloride infiltration. For decks with distress, the existing deck is typically milled and repaired with a low-slump concrete overlay as part of a more extensive bridge rehabilitation effort. For decks nearing replacement, asphaltic overlays maybe a cost effective option to improve ride quality. Refer to the following sections and Table 40.5-1 and Table 40.5-2 for a list of common overlay methods and additional information.

40.5.1.1 Thin Polymer Overlay

A thin polymer overlay (TPO) is expected to extend the service life of a bridge deck for 7 to 15 years. This overlay adds minimal dead load to the existing structure while providing an impermeable surface to prevent chlorides from infiltrating the deck. It can also be used to improve or restore friction on bridge decks.

In general, thin polymer overlays are defined as 1-inch thick or less overlays consisting of a polymer binder with aggregates and can be placed either as a multi-layer, slurry, or premixed system. Typical polymer binders are either epoxy, polyester, or methacrylate based. For WisDOT applications, TPO's consist of a two-layer, two-component epoxy polymer in conjunction with natural or synthetic aggregates for a 1/4-inch minimum total thickness. For dead load purposes, use 5 psf for thin polymer overlays. Refer to the approved products list for a list of pre-qualified polymer liquid binders.

Cracks will develop in a new concrete deck throughout the first couple of years in response to vehicular and environmental loads. Initial concrete cracking should occur within the first two years of new deck construction. Placement after this time allows the overlay to seal existing cracks and may reduce reflective cracking in the overlay. Therefore, the earliest a thin polymer overlay shall be placed on a new deck is the following construction season. If it is determined that a thin polymer overlay should be placed in the next construction season, the thin polymer overlay should be included in the same contract as the new deck.

Thin polymer overlays can be used in lieu of resealing the deck on a project-to-project basis with BOS approval. Approval occurs through the structure certification process. Some examples where TPOs might be used instead of deck sealing are where heavy snowmobile traffic is expected or when the safety certification provides justification for enhanced friction surface treatment. See 40.5.5.1 for deck sealing usage in place of thin polymer overlays.

Sufficient bond strength is critical in maximizing the overlay's service life. The bond strength can be reduced by poor surface preparations, traffic conditions, moisture, and distressed concrete. As a result, TPO's should be used based on the following restrictions:

- Recommended on decks with a NBI rating greater than 7 to help mitigate chloride infiltration. The deck should be in good condition with wearing surface distressed areas not exceeding 2% of the total deck area.

- Not recommended on decks that have been exposed to chlorides for more than 10 years old or with a NBI rating less than 7. These restrictions assume that significant chloride infiltration has already occurred. When a robust deck washing and sealing program has been used, TPO's may be placed on decks 10-15 years old with above average deck condition.

- TPO's should not be placed on Portland cement concrete patches less than 28 days old. Patch and crack repairs shall be compatible with the overlay material.

- The earliest a thin polymer overlay shall be placed on a new deck is the following construction season. If it is determined through structure certification that a thin polymer overlay should be placed in the next construction season in lieu of future deck sealing, the thin polymer overlay should be included in the same contract as the new deck.

- Use of TPO's on the concrete approaches should be avoided. Slab-on-grade conditions may cause the overlay to fail prematurely due to moisture issues.

- Not recommended on decks with widespread cracking, large cracks (>0.04 in), or active cracks (e.g. longitudinal reflective cracks between PS box girders). These cracks are likely to reflect through the overlay, even when fully repaired.

- Decks with an existing TPO may be considered for a TPO re-application provided that the previously discussed restrictions can be assumed to be satisfied. Generally, this assumes the existing overlay performed well over its expected service life and the effective deck exposure did not exceed 15 years, such that significant chloride

infiltration has not occurred. If signification chloride infiltration is expected, a re-application would not be recommended.

Thin polymer overlays may be considered where friction needs to be restored or improved. In most cases, the two-layer polymer overlay system should be used as it will improve surface friction and protect the deck against future chloride infiltration. For situations requiring a high skid resistance, calcined bauxite or other alternative aggerates may be considered in lieu of natural or synthetic aggregates.

40.5.1.2 Low Slump Concrete Overlay

A low slump concrete overlay, also referred to as a concrete overlay, is expected to extend the service life of a bridge deck for 15 to 20 years. This system is comprised of low slump Grade E concrete and has a 1-1/2 inch minimum thickness. The overlay thickness can accommodate profile and cross-slope differences, but typically does not exceed 4-1/2 inches. Thicker overlays become increasingly unpractical due to load and cost implications.

Low slump Grade E concrete requires close adherence to the specification, including equipment, consolidation, and curing requirements. A properly cured concrete overlay will help limit cracks, but inevitably the concrete overlay will crack. After the concrete overlay has been placed, it is beneficial to seal cracks in the overlay to minimize deterioration of the underlying deck. The overlay may require crack sealing the following year and periodically thereafter.

On delaminated but structurally sound decks, a rehabilitation concrete overlay is often the only alternative to deck replacement. Typically, prior to placing the concrete overlay a minimum of 1" of existing deck surface is removed along with any unsound material and asphaltic patches.

Rehabilitation concrete overlays are performed when significant distress of the wearing surface has occurred. If more than 25% of the wearing surface is distressed, an in-depth cost analysis should be performed to determine if a concrete overlay is cost effective verses a deck replacement.

The quantity of distress on the underside of deck or slab should be negligible, less than 5%, indicating that the bottom mat of reinforcement steel is not significantly deteriorated. If significant quantities of distress are present under the deck, a deck replacement may be required in the future; an overlay at this time might not achieve full service life, but may be placed to provide a good riding surface until replacement.

If the structure has an existing overlay, the overlay condition should be evaluated in addition to the other previously discussed considerations. If the concrete deck remains structurally sound, it may be practical to remove an existing overlay and place a new overlay before replacing the entire deck. Prior to placing the concrete overlay, the existing overlay should be removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

A polyester polymer concrete (PPC) is expected to extend the service life of a bridge deck for 20 to 30 years. This system is a mixture of aggregate, polyester polymer resin, and initiator; which can be placed as a deck overlay using conventional concrete mixing and placement equipment, albeit most likely dedicated to PPC usage. The main advantages of a PPC overlay is that it is impermeable and causes minimal traffic disruptions due to its quick cure time. High costs and lack of performance data are the main disadvantages.

Prior to the placement of the PPC overlay, a high molecular weight methacrylate (HMWM) binder is placed on the prepared deck. This bonds the overlay to the deck, and it also serves to seal existing cracks in the deck. When the existing concrete is in good condition, PPC is effective at mitigating chloride penetration due to its impermeability.

The total thickness of a PPC overlay is typically 3/4" to 1". While thicker overlays are possible, they are usually cost prohibitive. PPC can be placed at 3/4" thick as opposed to a typical 1 1/2" thick concrete overlay. This may help in situations where bridge ratings and/or profile adjustments are of concern.

Since most applications recommend a 1-inch or less overlay, PPC overlays are considered a thin polymer overlay and have similar requirements and restrictions. PPC overlays should be limited to decks in good condition that require shorter traffic disruptions for sites with high traffic volumes and lane closure restrictions. PPC is a durable product and has a relatively fast curing time (2 to 4 hours), but also has a higher cost as compared to a concrete overlay. PPC overlays should be used based on the following restrictions:

- Deck wearing surface distress should not exceed 5% of the total deck area.

- Decks should have a NBI rating of 7 or greater and be less than 15 years old. Older decks may be considered when the existing deck has been protected by a thin polymer overlay or when chloride testing indicates acceptable chloride levels at the reinforcement. Chloride contents at the reinforcement should not exceed 5 lbs/CY for decks with epoxy coated reinforcement. PPC overlays are not recommended on decks with uncoated top mat reinforcement. Decks exposed to chlorides, exceeding 10 years, should consider a ¾-inch minimum scarification to remove chlorides.

- PPC overlays should not be placed on concrete decks or Portland cement concrete patches less than 28 days, unless approved otherwise. Patch and crack repairs shall be compatible with the overlay material.

- PPC shall not be used for structural repairs due to costs and performance concerns.

Note: PPC overlays are expensive and new to WisDOT. As a result, use of PPC overlays should be limited to preservation projects that meet the requirements outlined in Figure 40.5-2 or as approved by the Bureau of Structures.

40.5.1.4 Polymer Modified Asphaltic Overlay

A polymer modified asphaltic (PMA) overlay is expected to extend the service life of a bridge deck for 10 to 15 years. This system is a mixture of aggregate, asphalt content, and a thermoplastic polymer modifier additive, which can easily be placed as a deck overlay using conventional asphalt paving equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

The added polymer allows for the overlay to resist water and chloride infiltration. Proper mix control and placement procedures are critical in achieving this protection. Core tests have shown the permeability of this product is dependent on the aggregate. As a result, limestone aggregates should not be used.

PMA overlays can be used on more flexible structures (e.g. timber decks or timber slabs) and to minimize traffic disruptions.

Designers should contact the region to determine if a PMA overlay is a viable solution for the project. In some areas, product availability or maintaining an acceptable temperature may be problematic.

Note: PMA overlays are expensive, have a limited service life relative other overlay types, and product availability may be problematic. As a result, PMA overlays usage should be limited.

40.5.1.5 Asphaltic Overlay

An asphaltic overlay, without a waterproofing membrane, is expected to extend the service life of a bridge deck for 3 to 7 years. This system may be a viable treatment if the deck or bridge is programmed for replacement within 4 years on lightly traveled roadways and is able to provide a smooth riding surface. Without a waterproofing material, the overlay may trap moisture at the existing deck surface, which may accelerate deck deterioration.

These overlays must be watched closely for distress as the existing deck surface problems are concealed. This system is typically an asphaltic pavement with a mixture of aggregates and asphaltic materials, which can easily be placed as a deck overlay using conventional asphaltic mixing and placement equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

Note: Asphaltic overlays, without a waterproofing membrane, are not eligible for federal funds.

40.5.1.6 Asphaltic Overlay with Waterproofing Membrane

An asphaltic overlay, with a waterproofing membrane, is currently being used on a very limited basis. This system is expected to extend the service life of a bridge deck for 5 to 15 years. Experience indicates that waterproofing membranes decrease the rate of deck deterioration by preventing or slowing the migration of water and chloride ions into the concrete.

In the 1990's, waterproofing membranes were actively used with asphaltic overlays for protecting existing decks, but were phased out by 2009 when they were restricted due to performance concerns and the inability to inspect the deck. As a result, low slump concrete or PMA overlays are currently recommended when deck or bridge replacements are programed beyond 4 years, unless approved otherwise.

Note: Asphaltic overlays, with a waterproofing membrane, requires prior-approval by the Bureau of Structures. This system is currently under review for possible improvements.

40.5.1.7 Other Overlays

Several other overlay systems have been used on past projects, but are generally not used currently. Use of these systems or other systems not previously mentioned require prior-approval by the Bureau of Structures.

- Micro-silica (silica-fume) modified concrete overlay – Provides good resistance to chloride penetration due to its low permeability.

- Latex modified concrete overlay – Provides a long-lasting overlay system with minimal traffic disruptions. Several other states are currently using this overlay method with hydrodemolition deck preparations.

- Reinforced concrete overlays:

    o Thin overlays (< 4 ½") – Uses a superplasticizer and fiber reinforcement (steel or synthetic) for additional crack control by reducing cracks and crack widths.

    o Thick overlays (≥ 4 ½") – Uses steel reinforcements, rebar or weld wire fabric, typically for new structural decks. This overlay is intended to provide at least one layer of steel reinforcement, in each direction, for crack control. This overlay is currently recommended for PS box girder superstructures, which allows for composite details and improved means to control longitudinal reflective cracking. For most cases, steel reinforcement is not required when rehabilitation overlays exceed 4 1/2 - inches. Use of low slump Grade E concrete may not be suitable when incorporating steel reinforcements.

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 21 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
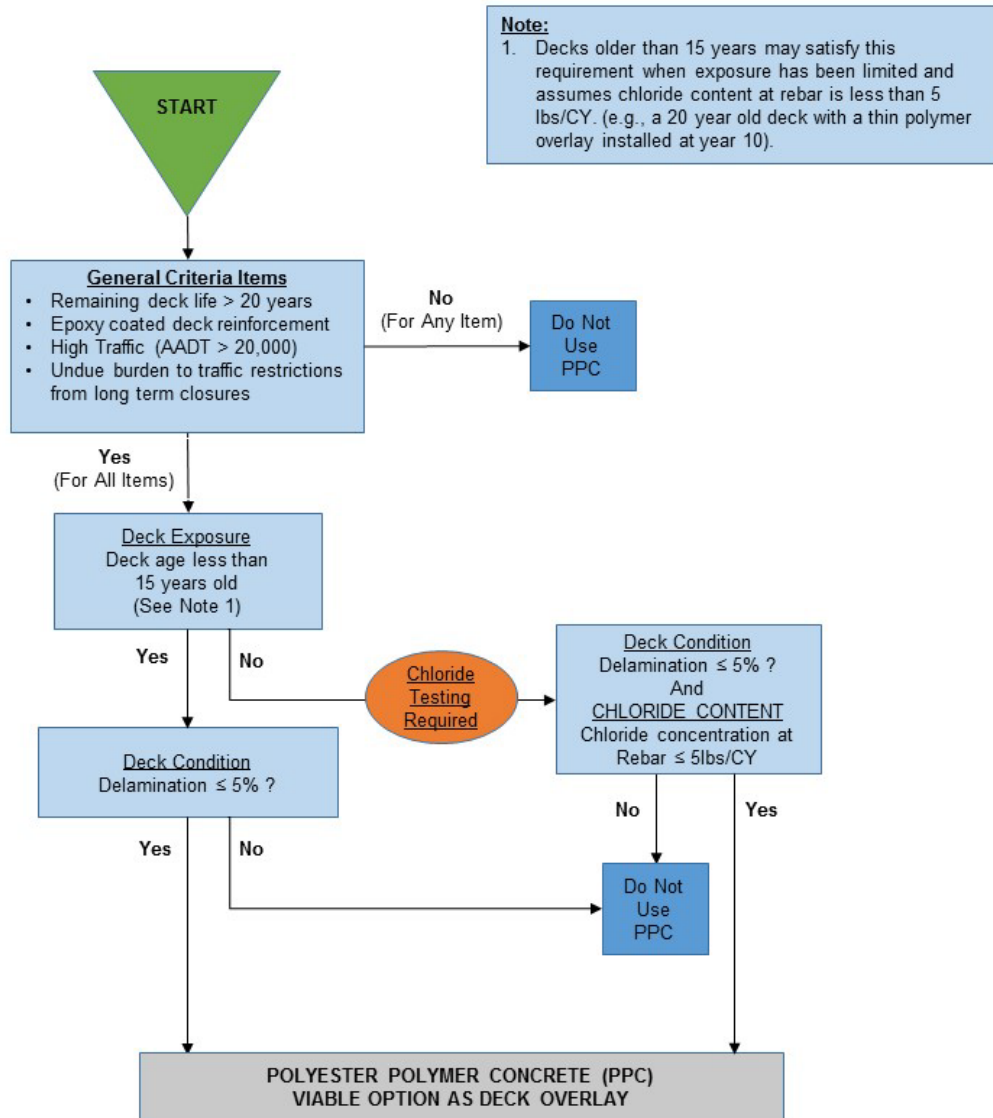
Figure 40.5-2
Polyester Polymer Concrete Overlay Usage Flowchart

40.5.3 Deck Assessment

The following are common deck assessment tools that can be used to survey existing deck conditions:

- Visual Inspections – Used to detect surface cracks, discontinuities, corrosion, and contamination.

- Audible Inspections - The two most common types of audible inspections are chain dragging and hammer sounding. Chain dragging is normally used on large concrete surface areas, such as bridge decks, while hammer sounding can be used on a number of materials in random locations. Both methods typically rely on the experience of the inspector to differentiate the relative sounds of similar materials.

- Infrared thermography - Infrared Thermography (IR) is an alternative tool for locating and mapping delaminations in bridge decks and pavements. A technique using an infrared scanner and control video camera, infrared thermography senses temperature differences between delaminated and non-delaminated areas.

- Ground penetrating radar (GPR) – GPR is a technique using electromagnetic signals, which can detect dielectric differences. This method can be used to measure concrete cover, overlay thickness, and reinforcing steel locations. This method can also be used to locate delaminations.

- Deck cores – Cores can be used to determine existing overly thicknesses, concrete cover, and concrete strength. As-built plans should only be used as a reference for existing conditions. Additionally, cores can be used to determine chloride content profiles. For asphaltic overlay, coring may be the best tool for deck assessments.

- Chloride Ion Testing - Chloride ions are the major cause of reinforcing steel corrosion in concrete. In evaluating chloride content, it is recommended that a chloride profile (chloride concentration percentage versus depth measurement below the concrete surface) be developed. This profile is important for assessing the future corrosion susceptibility of steel reinforcing and in determining the primary source of chlorides.

- Half-cell potential testing - A method used to detect whether the reinforcing steel is under active corrosion.

Visual inspections, audible inspections, and IR are the most common deck assessment tools for identifying delaminations and unsound concrete. For more information on deck assessment tools, refer to the Structure Inspection Manual – Part 5 – NDE and PDE Testing. Deck condition surveys should be placed on the structures plans. This should include the survey type and date when the survey was completed.

40.5.4 Deck Preparations

Prior to placing overlays, the existing deck surface will require deck preparations to repair the existing deck and to ensure that the overlay is properly bonded to the existing concrete. These preparations can range from sand blasting the entire deck to milling the entire deck with extensive repairs and are dependent on the existing deck conditions (distress, chloride concentration, existing overlay, proposed overlay, etc.).

The below deck preparations are typically used prior to placing overlays. Check the latest specifications for additional information.

Concrete Removal

Concrete deck removal usually includes the removal of unsound surface materials and the removal of a predetermined depth to remove concrete with high chloride concentrations. The following techniques can be used for large concrete removal areas:

Mechanical scarification or milling – The removal of existing deck to predetermined depth using a milling machine and other approved operations. This process can remove concrete with high chloride contents. However, this aggressive removal process has the potential to introduce micro-cracking into the existing deck.

Hydrodemolition – The removal of existing deck to a predetermined depth and the ability to selectively remove distressed areas using ultra high-pressure water-jetting (above 25,000 psi). A benefit to this process is that it does not introduce micro-cracking. WisDOT has very limited experience with this process and is usually cost prohibitive.

Generally, decks receiving a low slump concrete overlay will also include a 1-inch minimum deck removal. This assumes the existing top of deck has been exposed long enough to develop high chloride concentrations and would benefit from a milling operation. For early aged or protected (e.g. polymer overlay) decks, concrete milling may not be necessary prior to the overlay application and may be deferred to future overlay applications. Typically, only one aggressive milling operation is practical for a deck to leave sufficient cover for future overlays. Maintain ½" to 1" of rebar cover to ensure proper bonding and to protect the rebar and coating during the milling operation.

Deck Repairs

Care should be taken to limit damaging sound concrete and the existing reinforcement. Use of appropriate tools, hammers no more than 35 pounds and no more than 15 pounds when within one inch of the steel, is intended to limit distressed areas and avoid full-depth repairs. Additionally, saw cut depths should be carefully monitored such that the existing steel is not cut.

Cathodic protection may be warranted for decks with a high chloride content to help prevent corrosion from initiating.

The following items are associated with repairing distressed deck areas as shown in Figure 40.5-3:

Preparation Decks Type 1 – The removal of existing patches and unsound concrete only to a depth that exposes 1/2 of the peripheral area of the top or bottom bar steel in the top mat of reinforcement. Care should be taken to limit damaging sound concrete.

Preparation Decks Type 2 – The removal of existing unsound concrete below the limit of the type 1 removal described above. One inch below the bottom of the top or bottom bar steel in the top mat of reinforcement is the minimum depth of type 2 removal.

Full-Depth Deck Repair – The complete removal of existing concrete.

PREPRATION DECKS TYPE 1

PREPRATION DECKS TYPE 2

FULL-DEPTH DECK REPAIR
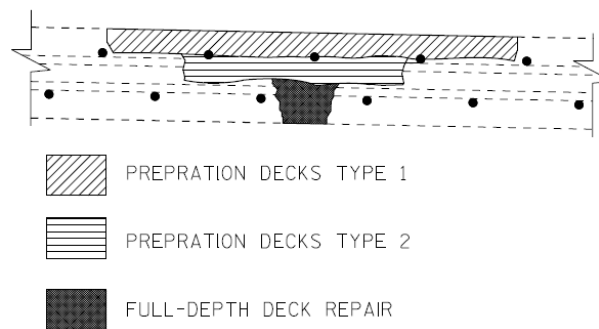
Figure 40.5-3
Deck Repairs

Deck Patches

Portland cement concrete is the preferred patch material. This material is easy to work with and very economical. When traffic impacts warrants, other materials may be considered. For concrete overlays, Type 1 and Type 2 deck patch repairs should be filled during the concrete overlay placement. Full-depth deck repairs should not be filled during the concrete overlay placement, but rather filled and curing a minimum of 24 hours before placing the concrete overlay. For other overlays, concrete repairs are usually properly cured prior to placing the overlay.

For minimal traffic impacts, a rapid-set material may be used for deck patches on asphaltic and thin polymer overlays. When repair quantities are minimal, distress areas less than 5% of the entire deck area, PPC overlays may use PPC to fill deck repairs prior overlay placement. See Table 40.5-1 for typical deck patch materials. Refer to the approved products list for a list of pre-qualified rapid setting concrete patch materials and their associated restrictions.

Surface Removal and Surface Preparation

Overlays require a properly prepared deck to achieve the desired bond strength. The following techniques are used for deck surface removal and preparations for an overlay:

Air cleaning – A preparation process to remove loose materials with compressed air. This process is intended to remove any material that may have gathered after the use of surface or concrete removal processes. This process is performed just prior to installing the overlay.

Water blasting (pressure or power washing) - A preparation process used to remove loose materials using low to high pressure water (5,000 psi to 10,000 psi). This process is beneficial as it keeps down dust and can remove loose particles.

Sand blasting – A surface removal process to remove loose material, foreign material, and loose concrete with sand material.

Shot blasting – A surface removal process to remove loose material, foreign material, and loose concrete by propelling steel shot against the concrete surface. This process also provides a roughen surface texture for improved bonding for overlays. Note: TPO's and PPC overlays provisions required a concrete surface profile meeting CSP-5 prior to overlay placement. This surface profile can be achieved using medium to medium-heavy shot blast.

40.5.5 Preservation Techniques

The following are some of the common activities being used to preserve decks and overlays:

- Deck cleaning (sweeping and power washing)

- Deck sealing/crack sealing

- Joint cleaning

- Joint repairs

- Deck patching

For additional preservation techniques and information refer to Chapter 42-Bridge Preservation.

40.5.5.1 Deck Sealing

Deck sealing has been found to be a cost-effective tool in preserving decks and overlays. In general, deck treatments should be applied as early as possible and re-applied thereafter. The frequency of deck sealing is dependent on the roadway traffic volume. Decks are to be sealed at initial construction and then resealed at the frequency shown in Table 40.5-3. Decks are to be resealed twice prior to applying a thin polymer overlay. Crack sealing should be considered as a potential combined treatment when deck sealing.

Roadway ADT	Deck Sealing Frequency
ADT < 2,500	4 – 5 years
2,500 <= ADT < 6,500	4 years
6,500 <= ADT < 15,000	3 years
ADT >= 15,000	2 years*

*In place of deck sealing, a thin polymer overlay is recommended within 2 years of deck construction. Use of the thin polymer overlay at this time will help minimize traffic impacts related to deck preservation work.

Table 40.5-3
Deck Sealing Frequency

Thin polymer overlays can be used in lieu of resealing the deck on a project-to-project basis with BOS approval. Approval occurs through the structure certification process. Some examples where TPOs might be used instead of deck sealing are where heavy snowmobile traffic is expected or when the safety certification provides justification for enhanced friction surface treatment.

40.5.6 Other Considerations

- Bridges with Inventory Ratings less than HS10 after rehabilitation shall not be considered for overlays, unless approved by the Bureau of Structures Design Section.

- Inventory and Operating Ratings shall be provided on the bridge rehabilitation plans.

- Verify the desired transverse cross slope with the Regions as they may want to use current standards.

- On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans. If more than 1/3 of the steel is exposed and the bar ends are not anchored, either adjacent spans must be shored or a special analysis and removal plan are required. Reinforcement shall be anchored using Portland cement concrete.

- Asphaltic overlays should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic.

- All full-depth repairs shall be made with Portland cement concrete.

- Joints and floor drains should be modified to accommodate the overlay

- Concrete chloride thresholds – Chloride content tests measure the chloride ion concentrations at various depths. Generally, research has shown initiation of corrosion

is expected when the chloride content is between 1 to 2 lbs/CY in concrete for uncoated bars and 7 to 12 lbs /CY for epoxy coated bars at the reinforcement. These limits are referred to as the threshold for corrosion. Threshold limits do not apply to stainless steel rebar.

When the chloride ion content is greater than 0.8 lbs/CY in concrete for uncoated bars and 5 lbs /CY for epoxy coated bars at the reinforcement depth, measures should be considered to limit additional chloride infiltration.

- See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

- Refer the standard details for the most current bid items.

- Overlay transitional areas should be used and coordinated when accommodating profile differences. These transitions are intended to improve ride quality and protect against snowplow damage. Ideally, transitions are placed such that the overlay thickness remains constant, which requires a tapered removal of the existing surface over a sufficient distance. For profile adjustments 1 1/2-inch or greater, transitional areas should consider a minimum taper rate of 1:250 for low-speed applications (RSD< 50 mph) and for high-speed applications up to a 1:400 taper rate. Typically, thicker profile adjustments are provided off the bridge deck and are coordinated by the roadway designer. For profile adjustments less than 1 1/2-inch, a minimum rate of 1:250 may be used regardless of the roadway design speed. For a 3/4-inch minimum PPC overlay, provide a 16-feet minimum transition length. For a 1/4-inch TPO overlay, a 3-feet minimum transition length is sufficient. See Chapter 40 Standards for additional guidance.

40.5.7 Past Bridge Deck Protective Systems

In the past, several bridge deck protective systems have been employed on the original bridge deck or while rehabilitating the existing deck as described in 17.8. The following systems have been used to protect bridge decks:

- Epoxy coated deck reinforcement – Prior to the 1980's, uncoated (black) bars were used throughout structures, including bridge decks. Criteria for epoxy coated reinforcement was first introduced in 1981 as a deck protective system. At first, usage was limited to the top mat of deck reinforcement. By 1987, coated bars were required in the top and bottom mats for high volume roadways (ADT > 5000). By 1991, coated bars were required for all State bridges and on some local bridges (ADT > 1000). Currently, use of epoxy coated deck reinforcement is required on all bridge decks.

- Asphaltic overlay with Membranes – Use of this overlay system was largely discontinued in the 1990's.

- High Performance Concrete (HPC) - Use of HPC has been limited to Mega Projects.

- Thin Polymer Overlays – Use of this overlay system is currently being used.

- Polyester Polymer Concrete Overlays – Use of this overlay system currently being used limitedly.

- Additional Concrete Cover – Use of additional clear cover (> 2 ½ inches) has been used on bridges with high volume and high truck traffic.

- Stainless steel deck reinforcement – Use of stainless steel has been very limited.

- Fiber reinforce polymer (FRP) deck reinforcement - Use of FRP reinforcement has only be used for experimental purposes.

As-built plans should be reviewed for past deck protective systems to assist with the appropriate rehabilitation measures.

40.5.8 Railings and Parapets

Overlays may decrease the parapet height when the existing overlay is not milled off and replaced in-kind. See Chapter 30-Railings for guidance pertaining to railings and parapets associated with rehabilitation structures projects.

__40.6 Deck Replacements__

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges (does not include local roadways over STN routes) eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

__Table 40.6-1__
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:

Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the FDM and FDM SDD 14b7 for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, replace existing intermediate concrete diaphragms with new steel diaphragms at existing diaphragm locations (i.e. don't add intermediate lines of diaphragms). See Chapter 19 Standard Details and Steel Diaphragm Insert Sheets for additional information. Existing concrete diaphragms, in good condition, that are full-depth to the bottom of the girder (typically located at the abutments and piers) shall not be removed for a deck replacement.

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 36", 45", 54" and 70" girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections' draped and undraped strand patterns.

The 36", 45", 54", and 70" girders in Chapter 40-Bridge Rehabilitation standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at $0.75f_{pu}$,

- A concrete haunch of 2-1/2",

- Slab thicknesses from Chapter 17-Superstructure - General,

- A future wearing surface of 20 psf,

- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads,

- 0.5" or 0.6" dia. strands (in accordance with the Standard Details),

- f'_c girder = 8,000 psi,

- f'_c slab = 4,000 psi, and

- Required f'_c girder at initial prestress < 6,800 psi

36" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	76	82
6'-6"	74	80
7'-0"	69	78
7'-6"	66	76
8'-0"	65	75
8'-6"	63	69
9'-0"	62	67
9'-6"	60	65
10'-0"	59	64
10'-6"	58	63
11'-0"	51	61
11'-6"	50	60
12'-0"	49	58

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	102	112
6'-6"	100	110
7'-0"	98	108
7'-6"	96	102
8'-0"	94	100
8'-6"	88	98
9'-0"	88	96
9'-6"	84	90
10'-0"	84	88
10'-6"	82	86
11'-0"	78	85
11'-6"	76	84
12'-0"	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	130	138
6'-6"	128	134
7'-0"	124	132
7'-6"	122	130
8'-0"	120	128
8'-6"	116	124
9'-0"	112	122
9'-6"	110	118
10'-0"	108	116
10'-6"	106	112
11'-0"	102	110
11'-6"	100	108
12'-0"	98	104

70" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	150*	160*
6'-6"	146*	156*
7'-0"	144*	152*
7'-6"	140*	150*
8'-0"	138*	146*
8'-6"	134*	142*
9'-0"	132*	140*
9'-6"	128*	136
10'-0"	126*	134
10'-6"	122	132
11'-0"	118	128
11'-6"	116	126
12'-0"	114	122

Table 40.7-1
Maximum Span Length vs. Girder Spacing

*For lateral stability during lifting these girder lengths will require pick up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the

pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.

40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, consideration shall be given to replacing the entire deck in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the life-cycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W" rather than 54"). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. *The girders used for widenings may be the latest Chapter 19-Prestressed Concrete sections designed to LRFD or the sections from Chapter 40-Bridge Rehabilitation designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.*

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet **LRFD [3.6.5]** (600 kip loading) as a widening is considered rehabilitation. Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development Section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don't add intermediate lines of diaphragms).

40.9 Superstructure Replacement

Various types of superstructure replacements include replacing prestressed girders in-kind, replacing slabs in-kind and replacing steel girders with prestressed girders or slabs. When considering replacement of a deck on steel girders, consideration of the cost of painting the structural steel should be included in the evaluation.

Approval is required from BOS for all superstructure replacement projects. To ensure that the cost of a superstructure replacement is warranted, the substructure should be in good condition. In general, the superstructure replacement should remain the same as the original design to better ensure that substructure reuse is practical. See 40.10 for considerations regarding substructure reuse criteria.

WisDOT policy item:

Provided that the substructure meets the criteria in 40.10, the superstructure may be replaced. The superstructure shall be designed to current LRFD criteria.

Reuse of the existing substructure is contingent on the fixity of the substructure units remaining the same. If the fixity is changed, the substructure must be evaluated per the design loading of the original structure.

With the substructure needing further evaluation for increased dead load and/or change in fixity, discuss with BOS the acceptability of the evaluation results prior to continuing with final design.

40.10 Substructure Reuse and Replacement

When practical, substructure reuse may be an acceptable alternative to replacing the entire bridge. However, reuse will require early coordination with BOS, engineering judgement, and will be evaluated on a project-by-project basis. This evaluation should determine if the substructure can be reused "as-is" with or without minor surface repairs, reused with major repairs and/or strengthening, or needs to be replaced.

In general, "as-is" reuse of substructures should be reserved for in-kind superstructure replacements with little to no change in geometry, fixity, and service dead loads. Additionally, substructures should be in good condition and only require minor surface repairs. If satisfied, evaluation of the existing substructure with the load rating methodology as discussed in 45.3.2 for an existing (in-service) bridge (e.g. LFR) may be acceptable. An example of this condition would be an in-kind slab superstructure replacement with a substructure that remains in good condition. For other conditions (i.e. reuse with major repairs and/or strengthening), the substructure should be evaluated with the current load rating methodology (LRFR) as discussed in 45.3.1.1 for new bridge construction. If substructure reuse is found to be not practical due the expensive repairs and/or excessive strengthening, the substructure should be completely replaced.

Approval is required from BOS for all substructure reuse projects.

Normally it is acceptable to assume that the original bridge design was done correctly, however pier caps, either for multi-columned piers or open pile bents, have occasionally been under-designed. Further investigation is warranted for pier caps with nominal shear stirrups, rather than stirrups that appear to be designed for the girder configuration, etc.

See 40.15 for more information on substructure inspection.

Additional guidance regarding substructure reuse can be found in the FHWA publication *Foundation Reuse for Highway Bridges*.

40.10.1 Substructure Rehabilitation

Substructure rehabilitation work can vary significantly from minor concrete surface repairs to major repairs that includes strengthening members.

40.10.1.1 Piers

Pier caps and/or columns/shafts may show signs of distress due to spalled concrete. The spalling may be completely around some of the longitudinal bar steel, thus destroying the bond. The concrete usually remains sound under the bearing plates, possibly due to compressive forces preventing salt intrusion and/or deterioration from freeze thaw cycles.

If the bond of the structural reinforcement is not compromised (at least half of the bar is bonded), rehabilitation measures include:

1. Concrete Surface Repair. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required for areas larger than nominal. See 40.20 for more information on FRP.

If the bond of the structural reinforcement is compromised (at least half of the bar is not bonded), rehabilitation measures include:

1. Concrete Surface Repair, with an option for Shotcrete for larger areas. Anchors may be required if depth of repair is excessive. Replacement reinforcement (or mesh) may be required if excessive rebar loss. Fiber Reinforced Polymer (FRP), either non-structural or structural, may be required. See 40.20 for more information on FRP.

40.10.1.2 Bearings

Bearings being replaced should follow the Chapter 27 Standard Details, as well as the Chapter 40 Standard for Expansion Bearing Replacement Details. Replace lubricated bronze bearings with either laminated elastomeric bearings (preferred, if feasible) or Stainless Steel TFE bearings. If only outside bearings are replaced, the difference in friction/resistance values between adjacent girders can be ignored. In addition to the bid item for the new bearing, the STSP Removing Bearings is required.

For bearings requiring maintenance, consider the SPV Cleaning and Painting Bearings. Special Provisions Bearing Maintenance and Bearing Repair may also be worthy of consideration.

40.11 Other Considerations

40.11.1 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.

40.11.2 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.11.3 Repairs to Prestressed Concrete Girders

Repairs to prestressed concrete girders over traffic lanes should provide measures to protect against concrete patches from coming loose. After repairing the concrete section, non-structural FRP is often used to confine and protect the repair area. Other measures to positively connect concrete patches, beyond the bond strength of the two surfaces, may be considered.

40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Recommended paint maintenance is determined with assistance from the Wisconsin Structures Asset Management System (WiSAMS), which utilizes information provided by the routine bridge inspections.

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects,

including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

Existing steel expansion devices shall be modified or replaced with watertight expansion devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6' or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide downhill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

40.14 Superstructure Inspection

40.14.1 Prestressed Girders

On occasion prestressed concrete girders are damaged during transport, placement, or as a result of vehicle impact. Damage inspection and assessment procedures are necessary to determine the need for traffic restrictions, temporary falsework for safety and/or strength. Three predominant assessment areas are damage to prestressing strands, damage to concrete, and remaining structural integrity.

Where damage to a girder results in any significant loss of concrete section, an engineering analysis should be made. This analysis should include stress calculations for the damaged girder with comparison to the original design stresses. These calculations will show the loss of strength from the damage.

Assessment of damage based on the loss of one or more prestressing strands or loss of prestress force is given as reason for restriction or replacement of girders. Some of the more common damages are as follows with the recommended maintenance action. However, an engineering assessment should be made on all cases.

1. If cracking and spalling are limited to the lower flange, patching is usually performed. This assessment should be based on calculations that may allow repair.

2. If cracking continues from flange into web, the girder is normally replaced. Findings indicate that sometimes repair-in-place may be the preferred decision.

3. Termini of cracks are marked, and if the cracks continue to grow the girder may be replaced. This is a good method to determine the effect of loads actually being carried.

4. When large areas of concrete are affected, or when concrete within stirrups is fractured, replace. At times it may be more appropriate to repair-in-place than replace.

Repair and/or replacement decisions are based on structural integrity. Load capacity is by far the most important rationale for selection of repair methods. Service load capacity needs to be calculated if repair-in-place is contemplated.

Evaluations of criteria for assessment of damage related to the two predominant areas associated with structural integrity are the following:

1. A structural analysis is made to determine the stresses in a damaged girder on the basis of the actual loads the girder will carry. This technique may result in a girder that has less capacity than the original, but can still safely carry the actual loads. This assessment results in consideration of possible repair-in-place methods which normally are cost effective.

    or

2. A structural analysis is made to determine the load capacity and rating of the girder. If the capacity and rating of the girder is less than provided by the original design, the girder shall be replaced. This assessment will provide a girder equal to the original design, but precludes possible repair-in-place methods that are normally less costly.

Location and size of all spalled and unsound concrete areas shall be recorded. Location, length, and width of all visible cracks shall be documented. All damage to prestress strands and reinforcing steel shall be reported. Location of hold-down devices in the girder shall be shown in relation to the damage. Horizontal and vertical misalignment along the length of the girder, and at points of localized damage, shall be reported. (These measurements might best be made by string-lining). Growth of cracks shall be monitored to determine that the cracked section has closed before extending to the web.

Critical damage is damage to concrete and/or the reinforcing elements of prestressed concrete girders such as:

1. Cracks extend across the bottom flange and/or in the web directly above the bottom flange. (This indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset is measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.

4. Vertical misalignment in excess of the normal allowable.

5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface. (This indicates permanent deformation of stirrups).

40.14.2 Steel Beams

These are three alternate methods of repairing damaged steel beams. They are:

1. Replace the total beam,

2. Replace a section of the beam, or

3. Straighten the beam in-place by heating and jacking.

The first alternate would involve removing the concrete deck over the damaged beam, remove the damaged section and weld in a new piece; then reconstruct the deck slab and railing over the new girder. Falsework support is required at the locations where the beams are cut and probably in the adjacent span due to an unbalanced condition.

The second alternate involves cutting out a section of the beam after placing the necessary supporting members. The support is placed using calibrated jacks. The section is cut out as determined by the damage. A new section plus any vertical stiffeners and section of cover plates would be welded in. This involves butt welds on both the flange and web. The welding of the web is difficult due to minor misalignments to start with plus the tendency of thin plates to move from the heat of welding.

The third alternate of heating and jacking the in-place beam to straighten it is a difficult procedure but can be done by personnel familiar and knowledgeable of the process. It is important to maintain heat control under 1300°F maximum. Use an optical pyrometer to determine heat temperature. There is no specified tolerance for the straightened member. The process is deemed satisfactory when a reasonable alignment is obtained.

Based on the three alternates available, the estimated costs involved and the resultant restoration of the beam to perform its load carrying capacity, heat straightening is a viable option in many cases.

The structural engineer who will be responsible for plan preparation should field review the site with the Regional Bridge Maintenance Engineer.

40.15 Substructure Inspection

The inspection of substructure components may reveal deteriorating concrete in areas exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows. Footings and pilings exposed due to erosion and undermining could result in loss of bearing capacity and/or section. Utilize HSIS data to flag potential scour concerns (code 6000), with scour defects in condition state 4 being a significant concern.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Original pile capacities are determined from plans, or if available, the pile driving records. Reuse of steel pile sections will require checking the remaining load carrying capacity if section loss is determined to be present. Steel piling should be checked:

- Immediately below the splash zone or water line for deterioration and possible loss of section. High section loss occurs in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line.

- Below abutments where the berm soil (material beneath riprap) has settled below the abutment bottom and water appears to be flowing from beneath the abutment or stream water has direct access to the piling.

If there is piling section loss or undermined spread footings, capacities of existing piling and/or footings will need to be recomputed for load rating purposes.

Timber substructure components may exhibit deterioration due to fungus decay, abrasion wear and weathering. Also, physical damage may be caused by vermin attacks, chemicals, fires, and collisions. Prior to reuse, timber backed abutments and pier bents shall be checked, by boring, for material and mechanical condition, section loss and structural adequacy. Generally, timber substructures are not good candidates for substructure reuse due to their limited service life.

Bearing condition needs to be evaluated. When possible, replacement expansion bearings should be laminated elastomeric bearings. Replacing expansion devices to reduce chloride infiltration is often warranted.

40.16 Concrete Anchors for Rehabilitation

Concrete anchors are used to connect concrete elements with other structural or non-structural elements and can either be cast into concrete (cast-in-place anchors) or installed after concrete has hardened (post-installed anchors). This section discusses post installed anchors used on bridge rehabilitation projects. Note: this section is also applicable for several cases where post installed anchors may be allowed in new construction.

This section includes guidance based on the ACI 318-14 manual, hereafter referred to as ACI. (AASHTO currently does not have guidance for anchors.)

40.16.1 Concrete Anchor Type and Usage

Concrete anchors installed in hardened concrete, post-installed anchors, typically fall into two main groups – adhesive anchors and mechanical anchors. For mechanical anchors, subgroups include undercut anchors, expansion (torque-controlled or displacement controlled) anchors, and screw anchors.

Mechanical anchors are seldom used for bridge rehabilitations and current usage has been restricted due to the following concerns: anchor installation (hitting rebar, abandoning holes, and testing), the number of different anchor types, design requirements that are more restrictive than adhesive anchors, the ability to remove and reuse railings/fences, and the collection of salt water within the hole. Note: mechanical anchors may be considered when it has been determined cast-in-place anchors or through bolts are cost prohibitive, adhesive anchors are not recommended, and the above concerns for mechanical anchors have been addressed. See post-installed anchor usage restrictions for additional information.

An Approved Products List addresses some of the concerns for creep, shrinkage, and deterioration under load and freeze-thaw cycles for adhesives anchors. Bridge rehabilitations projects typically use adhesive anchors for abutment and pier widenings. Other bridge rehabilitation applications may also warrant the use of adhesive anchors when required to anchor into existing concrete. Refer to the Standards for several examples of anchoring into existing concrete.

In limited cases, post installed concrete anchors may be allowed for new construction. One application is the allowance for the contractor to use adhesive anchors in lieu of cast-in-place concrete anchors for attaching pedestrian railings/fencing. Refer to Chapter 30 Standards for pedestrian railings/fencing connections.

The following is a list of current usage restrictions for post installed anchors:

Usage Restrictions:

- Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers.

- **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column).**

- Adhesive anchors installed in the overhead or upwardly inclined position and/or under sustained tension loads shall not be used.

- The department has placed a moratorium on mechanical anchors. Usage is subject to prior-approval by the Bureau of Structures.

40.16.1.1 Adhesive Anchor Requirements

For adhesive anchors, there are two processes used to install the adhesive. One option uses a two-part adhesive that is mixed and poured into the drilled hole. The second option pumps a two-part adhesive into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. With either process, the hole must be properly cleaned and a sufficient amount of adhesive must be used so that the hole is completely filled with adhesive when the rebar or bolt is inserted. The adhesive bond stresses, as noted in Table 40.16 1, are determined by the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 or ACI 355.4.

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 6 times the anchor diameter. The maximum embedment depth for is 20 times the anchor diameter.

The manufacturer and product name of adhesive anchors used by the contractor must be on the Department's approved product list for "Concrete Adhesive Anchors".

Refer to the *Standard Specifications* for additional requirements.

40.16.1.2 Mechanical Anchor Requirements

The required minimum anchor spacing is 6 times the anchor diameter. The minimum edge distance is 10 times the anchor diameter. The minimum member is the great of the embedment depth plus 4 inches and 3/2 of the embedment depth. ***Mechanical anchors are currently not allowed.***

40.16.2 Concrete Anchor Reinforcement

Reinforcement used to transfer the full design load from the anchors into the structural member is considered anchor reinforcement. **ACI [17.4.2.9]** and **ACI [17.5.2.9]** provide guidance for designing anchor reinforcement. When anchor reinforcement is used, the design strength of the anchor reinforcement can be used in place of concrete breakout strength per 40.16.3 and 40.16.4. Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load is considered to be supplementary reinforcement.

Per **ACI [2.3],** concrete anchor steel is considered ductile if the tensile test elongation is at least 14 percent and reduction in area is at least 30 percent. Additionally, steel meeting the

requirements of ASTM A307 is considered ductile. Steel that does not meet these requirements is considered brittle. Rebar used as anchor steel is considered ductile.

40.16.3 Concrete Anchor Tensile Capacity

Concrete anchors in tension fail in one of four ways: steel tensile rupture, concrete breakout, pullout strength of anchors in tension, or adhesive bond. The pullout strength of anchors in tension only applies to mechanical anchors and the adhesive bond only applies to adhesive anchors. Figure 40.16-1 shows the concrete breakout failure mechanism for anchors in tension.

The minimum pullout capacity (Nominal Tensile Resistance) of a single concrete anchor is determined according to this section; however, this value is only specified on the plan for mechanical anchors. The minimum pullout capacity is not specified on the plan for adhesive anchors because the anchors must be designed to meet the minimum bond stresses as noted in Table 40.16-1. If additional capacity is required, a more refined analysis (i.e., anchor group analysis) per the current version of ACI 318-14 Chapter 17 is allowable, which may yield higher capacities.
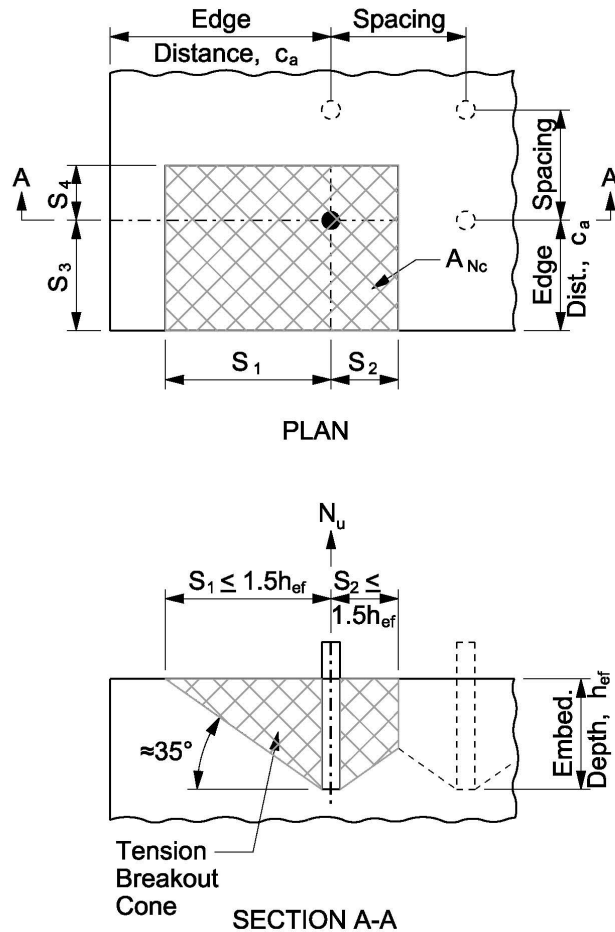
Figure 40.16-1
Concrete Breakout of Concrete Anchors in Tension

The projected concrete breakout area, A_{Nc}, shown in Figure 40.16-1 is limited in each direction by S_i:

  S_i = Minimum of:

    1. 1.5 times the embedment depth (h_{ef}),

    2. Half of the spacing to the next anchor in tension, or

    3. The edge distance (c_a) (in).

Figure 40.16-2 shows the bond failure mechanism for concrete adhesive anchors in tension.
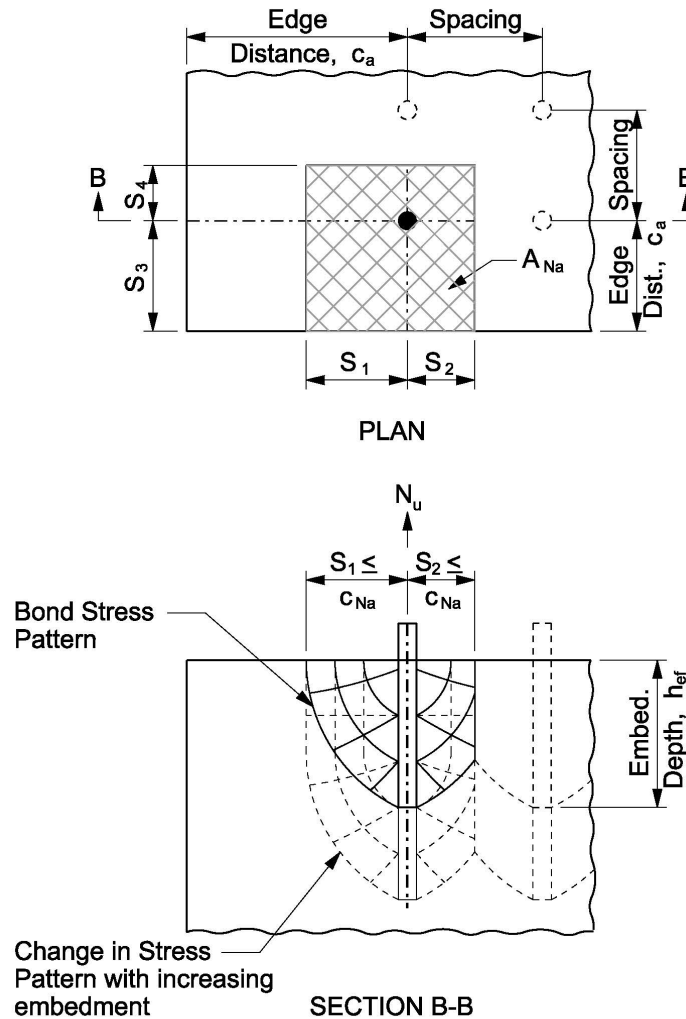
Figure 40.16-2
Bond Failure of Concrete Adhesive Anchors in Tension

The projected influence area of a single adhesive anchor, A_{Na}, is shown in Figure 40.16-2. Unlike the concrete breakout area, it is not affected by the embedment depth of the anchor. A_{Na} is limited in each direction by S_i:

S_i = Minimum of:

1. $c_{Na} = 10d_a\sqrt{\dfrac{\tau_{uncr}}{1100}}$,

2. Half of the spacing to the next anchor in tension, or

3.   The edge distance (c_a) (in).

Anchor Size, d_a	Adhesive Anchors			
	Dry Concrete		Water-Saturated Concrete	
	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	990	460	370	280
#5 or 5/8"	970	460	510	390
#6 or 3/4"	950	490	500	410
#7 or 7/8"	930	490	490	340
#8 or 1"	770	490	600	340

Table 40.16-1
Tension Design Table for Concrete Anchors

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for "Concrete Adhesive Anchors". The designer shall determine whether the concrete adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r. <u>For mechanical anchors:</u>

$$N_r = \phi_{ts}N_{sa} \leq \phi_{tc}N_{cb} \leq \phi_{tc}N_{pn}$$

In which:

ϕ_{ts}	=	Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
	=	0.65 for brittle steel as defined in 40.16.1.1
	=	0.75 for ductile steel as defined in 40.16.1.1
N_{sa}	=	Nominal steel strength of anchor in tension, **ACI [17.4.1.2]**
	=	$A_{se,N}f_{uta}$
$A_{se,N}$	=	Effective cross-sectional area of anchor in tension (in^2)
f_{uta}	=	Specified tensile strength of anchor steel (psi)

$\leq \quad 1.9f_{ya}$

$\leq \quad 125$ ksi

f_{ya} = Specified yield strength of anchor steel (psi)

ϕ_{tc} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.2
= 0.75 for anchors with supplementary reinforcement per 40.16.2

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**

$$= \frac{A_{Nc}}{9(h_{ef})^2} \psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

A_{Nc} = Projected concrete failure area of a single anchor, see Figure 40.16-1
= $(S_1 + S_2)(S_3 + S_4)$

h_{ef} = Effective embedment depth of anchor per Table 40.16-1. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.

$\psi_{ed,N}$ = Modification factor for tensile strength based on proximity to edges of concrete member, **ACI [17.4.2.5]**
= 1.0 if $c_{a,min} \geq 1.5h_{ef}$

$$= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef}$$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 (in)

$\psi_{c,N}$ = Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.2.6]**
= 1.0 when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels
= 1.4 when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

$$= \quad 1.0 \text{ if } c_{a,min} \geq c_{ac}$$

$$= \quad \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}} \text{ if } c_{a,min} < c_{ac}$$

c_{ac} = Critical edge distance (in)

= $4.0h_{ef}$

N_b = Concrete breakout strength of a single anchor in tension in uncracked concrete, **ACI [17.4.2.2]**

= $0.538\sqrt{f'_c}(h_{ef})^{1.5}$ (kips)

N_{pn} = Nominal pullout strength of a single anchor in tension, **ACI [17.4.3.1]**

= $\psi_{c,P}N_p$

$\psi_{c,P}$ = Modification factor for pullout strength of anchors based on the presence or absence of cracks in concrete, **ACI [17.4.3.6]**

= 1.4 where analysis indicates no cracking at service load levels

= 1.0 where analysis indicates cracking at service load levels

N_p = Nominal pullout strength of a single anchor in tension based on the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC193 / ACI 355.2

For adhesive anchors:

$$N_r = \phi_{ts}N_{sa} \leq \phi_{tc}N_{cb} \leq \phi_{tc}N_a$$

In which:

N_{cb} = Nominal concrete breakout strength in tension, **ACI [17.4.2.1]**

$$= \quad \frac{A_{Nc}}{9(h_{ef})^2} \psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

h_{ef} = Effective embedment depth of anchor. May be reduced per **ACI [17.4.2.3]** when anchor is located near three or more edges.

≤ $20d_a$ (in)

d_a = Outside diameter of anchor (in)

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in

uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.2.7]**

$=$ 1.0 if $C_{a,min} \geq C_{ac}$

$=$ $\dfrac{c_{a,min}}{c_{ac}} \geq \dfrac{1.5h_{ef}}{c_{ac}}$ if $C_{a,min} < C_{ac}$

$C_{a,min}$ $=$ Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 or Figure 40.16-2 (in)

c_{ac} $=$ Critical edge distance (in)

       $=$ $2.0h_{ef}$

N_a $=$ Nominal bond strength of a single anchor in tension, **ACI [17.4.5.1]**

       $=$ $\dfrac{A_{Na}}{4c_{Na}^{2}} \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$

A_{Na} $=$ Projected influence area of a single adhesive anchor, see Figure 40.16-2

       $=$ $(S_1 + S_2)(S_3 + S_4)$

$\Psi_{ed,Na}$ $=$ Modification factor for tensile strength of adhesive anchors based on the proximity to edges of concrete member, **ACI [17.4.5.4]**

       $=$ 1.0 if $C_{a,min} \geq C_{Na}$

       $=$ $0.7 + 0.3\dfrac{c_{a,min}}{c_{Na}}$ if $C_{a,min} < C_{Na}$

c_{Na} $=$ Projected distance from center of anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor

       $=$ $10d_a\sqrt{\dfrac{\tau_{uncr}}{1100}}$ (in)

τ_{uncr} $=$ Characteristic bond stress of adhesive anchor in uncracked concrete, see Table 40.16-1

$\Psi_{cp,Na}$ $=$ Modification factor for pullout strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [17.4.5.5]**

       $=$ 1.0 if $C_{a,min} \geq C_{ac}$

$$= \quad \frac{c_{a,min}}{c_{ac}} \geq \frac{c_{Na}}{c_{ac}} \text{ if } c_{a,min} < c_{ac}$$

N_{ba}　　= 　Bond strength in tension of a single adhesive anchor, **ACI [17.4.5.2]**

　　　　= 　$\tau_{cr} \pi d_a h_{ef}$

τ_{cr}　　= 　Characteristic bond stress of adhesive anchor in cracked concrete, see Table 40.16-1

Note: Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ICC-ES AC308 / ACI 355.4. For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr}.

In addition to the checks listed above for all adhesive anchors, the factored sustained tensile force must be less than or equal to the factored sustained tensile resistance per **ACI [17.3.1.2]**:

　　　0.50 $\phi_{tc} N_{ba} \geq N_{ua,s}$

40.16.4 Concrete Anchor Shear Capacity

Concrete anchors in shear fail in one of three ways: steel shear rupture, concrete breakout, or concrete pryout. Figure 40.16-3 shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in Figure 40.16-3 is limited vertically by H, and in both horizontal directions by S_i:

　　　H = Minimum of:

　　　　　1.　The member depth (h_a) or

　　　　　2.　1.5 times the edge distance (c_{a1}) (in).

　　　S_i = Minimum of:

　　　　　1.　Half the anchor spacing (S),

　　　　　2.　The perpendicular edge distance (c_{a2}), or

　　　　　3.　1.5 times the edge distance (c_{a1}) (in).
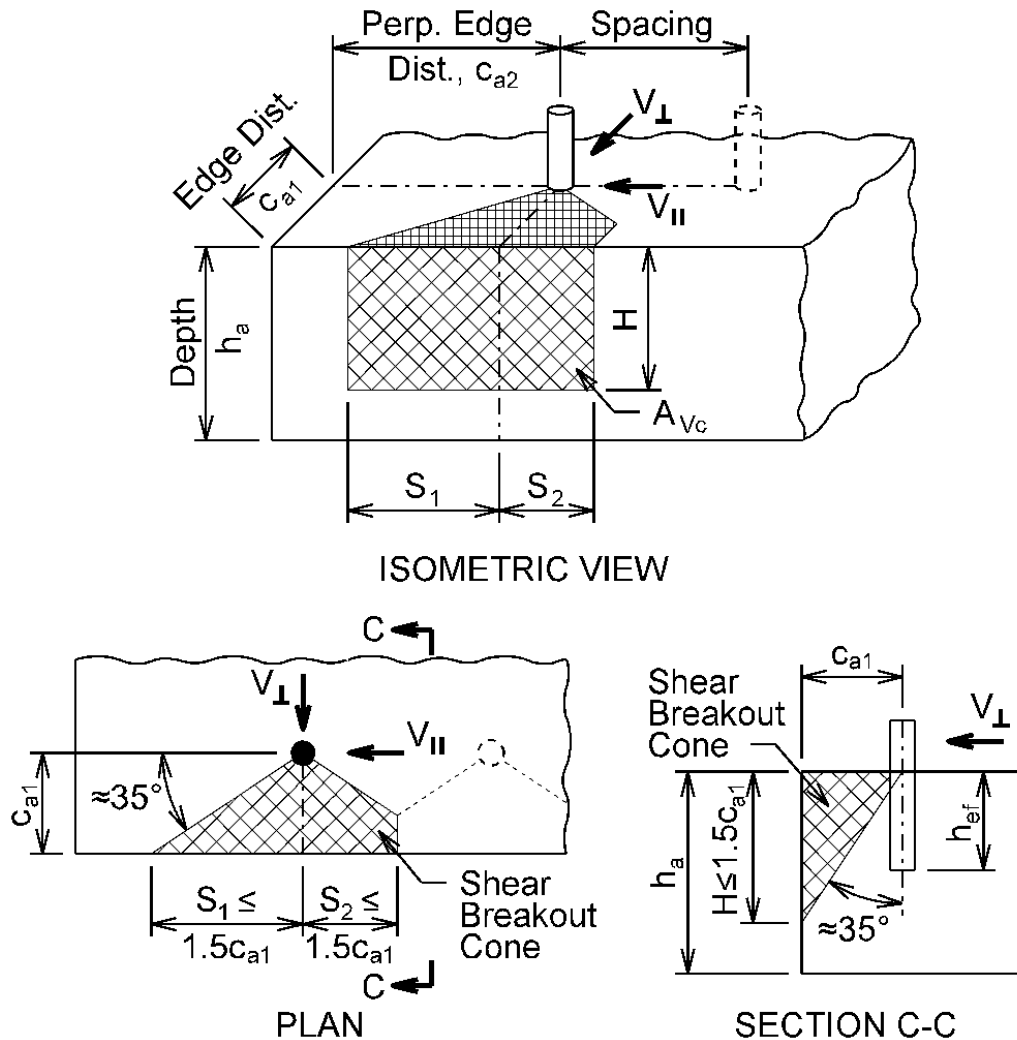
Figure 40.16-3
Concrete Breakout of Concrete Anchors in Shear

If the shear is applied to more than one row of anchors as shown in Figure 40.16-4, the shear capacity must be checked for the worst of the three cases. If the row spacing, SP, is at least equal to the distance from the concrete edge to the front anchor, E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the back anchor is checked for the full shear load. If the row spacing, SP, is less than the distance from the concrete edge to the front anchor, E1, then check Case 3. In case 3, the front anchor is checked for the full shear load. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.
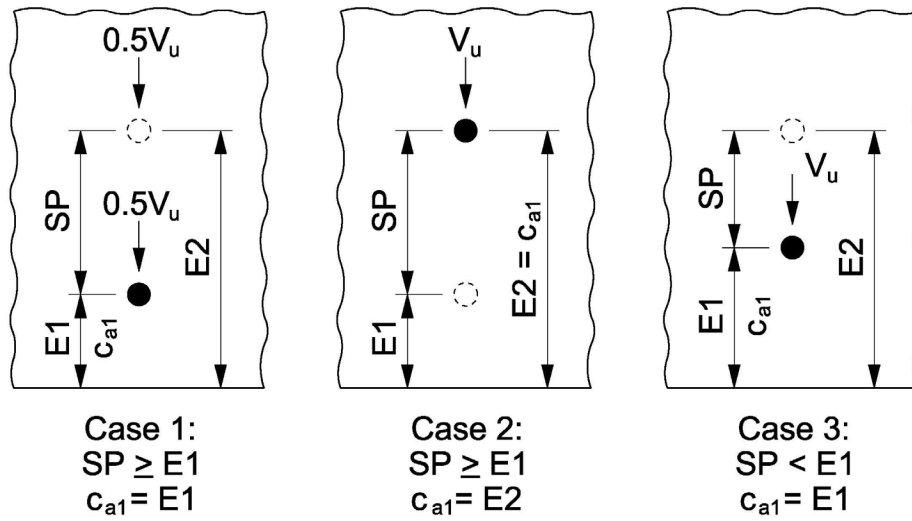
Figure 40.16-4
Concrete Anchor Shear Force Cases

The factored shear force on each anchor, V_u, must be less than or equal to the factored shear resistance, V_r. <u>For mechanical and adhesive anchors:</u>

$$V_r = \phi_{vs} V_{sa} \le \phi_{vc} V_{cb} \le \phi_{vp} V_{cp}$$

In which:

ϕ_{vs}	=	Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
	=	0.60 for brittle steel as defined in 40.16.1.1
	=	0.65 for ductile steel as defined in 40.16.1.1
V_{sa}	=	Nominal steel strength of anchor in shear, **ACI [17.5.1.2]**
	=	$0.6\, A_{se,V} f_{uta}$
$A_{se,V}$	=	Effective cross-sectional area of anchor in shear (in^2)
ϕ_{vc}	=	Strength reduction factor for anchors in concrete, **ACI [17.3.3]**
	=	0.70 for anchors without supplementary reinforcement per 40.16.2
	=	0.75 for anchors with supplementary reinforcement per 40.16.2
V_{cb}	=	Nominal concrete breakout strength in shear, **ACI [17.5.2.1]**
	=	$\dfrac{A_{Vc}}{4.5(c_{a1})^2} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{p,V} V_b$

A_{Vc} = Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.16-3

= $H(S_1 + S_2)$

c_{a1} = Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.16-3 and Figure 40.16-4 (in)

$\psi_{ed,V}$ = Modification factor for shear strength of anchors based on proximity to edges of concrete member, **ACI [17.5.2.6]**

= 1.0 if $c_{a2} \geq 1.5c_{a1}$ (perpendicular shear)

= $0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$ if $c_{a2} < 1.5c_{a1}$ (perpendicular shear)

= 1.0 (parallel shear)

c_{a2} = Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, see Figure 40.16-3 (in)

$\psi_{c,V}$ = Modification factor for shear strength of anchors based on the presence or absence of cracks in concrete and the presence or absence of supplementary reinforcement, **ACI [17.5.2.7]**

= 1.4 for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels

= 1.0 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels without supplementary reinforcement per 40.16.2 or with edge reinforcement smaller than a No. 4 bar

= 1.2 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge

= 1.4 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at no more than 4 inches

$\psi_{h,V}$ = Modification factor for shear strength of anchors located in concrete members with $h_a < 1.5_{ca1}$, **ACI [17.5.2.8]**

= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

h_a = Concrete member thickness in which anchor is located measured parallel to anchor axis, see Figure 40.16-3 (in)

$\Psi_{p,V}$ = Modification factor for shear strength of anchors based on loading direction, **ACI [17.5]**

= 1.0 for shear perpendicular to the concrete edge, see Figure 40.16-3

= 2.0 for shear parallel to the concrete edge, see Figure 40.16-3

V_b = Concrete breakout strength of a single anchor in shear in cracked concrete, per **ACI [17.5.2.2]** , shall be the smaller of:

$$[7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a}]\sqrt{f'_c}(c_{a1})^{1.5} \text{ (lb)}$$

Where:
$l_e = h_{ef} \leq 8d_a$
d_a = Outside diameter of anchor (in)
f'_c = Specified compressive strength of concrete (psi)

and

$$9\sqrt{f'_c}(c_{a1})^{1.5}$$

ϕ_{vp} = Strength reduction factor for anchors in concrete

= 0.65 for anchors without supplementary reinforcement per 40.16.2

= 0.75 for anchors with supplementary reinforcement per 40.16.2

V_{cp} = Nominal concrete pryout strength of a single anchor, **ACI [17.5.3.1]**

= $2.0N_{cp}$

Note: The equation above is based on $h_{ef} \geq 2.5$ in. All concrete anchors must meet this requirement.

N_{cp} = Nominal concrete pryout strength of an anchor taken as the lesser of:

mechanical anchors: $\dfrac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$

adhesive anchors: $\dfrac{A_{Na}}{4(c_{Na})^2}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$

and

$$\frac{A_{Nc}}{9(h_{ef})^2} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$$

For shear in two directions, check both the parallel and the perpendicular shear capacity. For shear on an anchor near a corner, check the shear capacity for both edges and use the minimum.

40.16.5 Interaction of Tension and Shear

For anchors that are subjected to tension and shear, interaction equations must be checked per **ACI [17.6]**.

If $\frac{V_{ua}}{\phi V_n} \leq 0.2$ for the governing strength in shear, then the full strength in tension is permitted:

$\phi N_n \geq N_{ua}$. If $\frac{N_{ua}}{\phi N_n} \leq 0.2$ for the governing strength in tension, then the full strength in shear is

permitted: $\phi V_n \geq V_{ua}$. If $\frac{V_{ua}}{\phi V_n} > 0.2$ for the governing strength in shear and $\frac{N_{ua}}{\phi N_n} > 0.2$ for the

governing strength in tension, then:

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$

40.16.6 Plan Preparation

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance as determined in 40.16.3.

Typical notes for bridge plans (shown in all capital letters):

Adhesive anchors located in uncracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR).   EMBED XX" IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive anchors located in cracked concrete:

ADHESIVE ANCHORS X/X-INCH (or No. X BAR).   EMBED XX" IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

When using anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item "Adhesive Anchors _-Inch".

For anchors using rebar, the rebar should be listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS Coated Structures".

When adhesive anchors are used as an alternative anchorage the following note should be included in the plans:

ADHESIVE ANCHORS SHALL CONFORM TO SECTION 502.2.12 OF THE STANDARD SPECIFICATION. *(Note only applicable when the bid item Adhesive Anchor is not used).*

It should be noted that AASHTO is considering adding specifications pertaining to concrete anchors. This chapter will be updated once that information is available.

40.17 Plan Details

1. Excavation for Structures on Overlays

    There is considerable confusion on when to use or not use the bid item "Excavation for Structures" on overlay projects. In order to remove the confusion, the following note is to be added to all overlay projects that only involve removal of the paving block (or less).

    > Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item "(insert applicable bid item)".

    If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay, the "Excavation for Structures" bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements, show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

    Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by current standard of a 0.02 ft/ft cross slope, a cross slope of 0.01 ft/ft or 0.015 ft/ft may be the most desirable.

    The designer should evaluate 3 types of repairs. "Preparation Decks Type 1" is concrete removal to the top of the bar steel. "Preparation Decks Type 2" is concrete removal below the bar steel. "Full Depth Deck Repair" is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of "Full Depth Deck Repair" on the plan sheet.

    Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction, consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

    The Region should provide the designer with a Rehabilitation Structure Survey Report that provides a complete description of the rehabilitation and estimated quantities. Contact the Region for clarifications on the scope of work.

    Additional items:

- Provide deck survey outlining areas of distress (if available). These plans will serve as documentation for future rehabilitations.

- Distressed areas should be representative of the surveyed areas of distress. Actual repairs will likely be larger than the reported values while removing all unsound materials.

- Provide Preparation Deck Type 1 & 2 and Full-Depth Repair estimates for areas of distress.

- Coordinate asphaltic materials with the Region and roadway designers.

See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

40.18 Retrofit of Steel Bridges

Out of plane bending stresses may occur in steel bridges that introduce early fatigue cracks or worse yet brittle fractures. Most, if not all, problems are caused at connections that are either too flexible or too rigid. It is important to recognize the correct condition as retrofitting for the wrong condition will probably make it worse.

40.18.1 Flexible Connections

A connection is too flexible when minor movement can occur in a connection that is designed to be rigid. Examples are transverse or bearing stiffeners fitted to a tension flange, floorbeams attached to the web only and not both flanges.

The solution for stiffeners and transverse connection plates is to shop weld the stiffener to the web and flanges. This becomes a Category C weld but is no different than the terminated web to stiffener weld. If the condition exists in the field, welding is not recommended. Retrofit is to attach a T-section with four bolts to the stiffener and four bolts to the flange. Similar details should be used in attaching floorbeams to the girder.

40.18.2 Rigid Connections

A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.

Other partial constraint details are:

1. Intersecting welds

2. Gap size-allowing local yielding

3. Weld size

4. Partial penetration welds versus fillet welds

5. Touching and intersecting welds

The solution is to create spaces large enough (approximately 1/4" or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4" and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

40.19 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

Effective Span Ft-In	T=Slab Thickness Inches	Transverse Bars & Spacing	Longitudinal Bars & Spacing	Longitudinal* Continuity Bars & Spacing
4-0	6.5	#5 @ 8"	#4 @ 8.5"	#5 @ 7.5"
4-3	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-6	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-9	6.5	#5 @ 7"	#4 @ 7.5"	#5 @ 7.5"
5-0	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-3	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-6	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
5-9	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
2-3	7	#4 @ 9"	#4 @ 11"	#5 @ 6.5"
2-6	7	#4 @ 8.5"	#4 @ 11"	#5 @ 6.5"
2-9	7	#4 @ 8"	#4 @ 11"	#5 @ 6.5"
3-0	7	#4 @ 7.5"	#4 @ 11"	#5 @ 6.5"
3-3	7	#4 @ 7"	#4 @ 11"	#5 @ 6.5"
3-6	7	#4 @ 6.5"	#4 @ 11"	#5 @ 6.5"
3-9	7	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6.5"
4-0	7	#4 @ 6"	#4 @ 10"	#5 @ 6.5"
4-3	7	#5 @ 9"	#4 @ 9.5"	#5 @ 7"
4-6	7	#5 @ 8.5"	#4 @ 9"	#5 @ 7"
4-9	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
5-0	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
4-3	7	#5 @ 7.5"	#4 @ 8"	#5 @ 7"
5-6	7	#5 @ 7'	#4 @ 7"	#5 @ 7"
5-9	7	#5 @ 7"	#4 @ 7"	#5 @ 7"
6-0	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-3	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-6	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
6-9	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
7-0	7	#5 @ 6"	#4 @ 6"	#5 @ 6"
4-0	7.5	#4 @ 7"	#4 @ 10.5"	#5 @ 6"
4-3	7.5	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6"
4-6	7.5	#4 @ 6.5"	#4 @ 10"	#5 @ 6"
4-9	7.5	#4 @ 6"	#4 @ 10"	#5 @ 6"
5-0	7.5	#5 @ 9"	#4 @ 9.5"	#5 @ 6"
5-3	7.5	#5 @ 8.5"	#4 @ 9"	#5 @ 6.5"

5-6	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
5-9	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
6-0	7.5	#5 @ 7.5"	#4 @ 8"	#5 @ 6.5"
6-3	7.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 6.5"
6-6	7.5	#5 @ 7"	#4 @ 7.5"	#5 @ 6.5"
6-9	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-0	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-3	7.5	#5 @ 6.5"	#4 @ 6.5"	#5 @ 6.5"
7-6	7.5	#5 @ 6.5"	#5 @ 10"	#5 @ 6.5"
7-9	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-0	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-3	7.5	#5 @ 6"	#5 @ 9.5"	#5 @ 6.5"

Table 40.19-1

Reinforcing Steel for Deck Slabs on Girders for Deck Replacements – HS20 Loading

Max. Allowable Design Stresses: f_c' = 4000 psi, f_y = 60 ksi, Top Steel 2-1/2" Clear, Bottom Steel 1-1/2" Clear, Future Wearing Surface = 20 lbs/ft. Transverse and longitudinal bars shown in table are for one layer only. Place identical steel in both top and bottom layer, except in negative moment region. Use in top layer for slab on steel girders in negative moment region when not designed for negative moment composite action.

40.20 Fiber Reinforced Polymer (FRP)

40.20.1 Introduction

Fiber reinforced polymer (FRP) material is a composite composed of fibers encased in a polymer matrix. The fibers provide tensile strength while the resin protects the fibers and transfers load between them. FRP can be used to repair or to retrofit bridges. Repair is often defined as returning a member to its original condition after damage or deterioration while retrofitting refers to increasing the capacity of a member beyond its original capacity.

For plan preparations, FRP repairs and retrofits are categorized as either structural strengthening or non-structural protection. Contact the Bureau of Structures Design Section for current Special Provisions and for other FRP considerations.

40.20.2 Design Guidelines

While there is no code document for the design of FRP repairs and retrofits, there are two nationally recognized design guidelines: the *Guide Specification for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (14.) hereinafter referred to as the AASHTO FRP Guide, and the *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures ACI 440.2R-08 (15.)* hereinafter referred to as the ACI FRP Guide.

Note: BOS has been evaluating the design methodologies found in the AASHTO FRP Guide and ACI RFP Guide. Noticeable differences between the guides warrants further investigation, with input from industry representation. FRP repairs and retrofits shall be in accordance with the applicable Special Provisions.

40.20.3 Applicability

Not all concrete structures can be retrofitted or repaired using FRP. Most FRP research has been conducted on normal sized members, therefore many of the design equations cannot be used with exceptionally large or deep members. Additionally, members with disturbed regions (D-regions) such as deep beams and corbels are outside of the scope of many design equations.

The structure must have some amount of load carrying capacity prior to the installation of the FRP. Due to the potential for premature debonding, FRP cannot be counted on to carry service loads; it may only be used increase the ultimate capacity of the structure for strength and extreme event load cases. The unrepaired or unretrofitted structure be able to carry the service dead and live loads:

$$R_r \geq \eta_i[(DC + DW) + (LL + IM)]$$

Where:

   R_r = factored resistance computed in accordance with AASHTO LRFD Section 5

η_i = load modifier = 1.0

DC = force effects due to components and attachments

DW = force effects due to wear surfaces and utilities

LL = force effects due to live load

IM = force effects due to dynamic load allowance

If capacity is added in flexure to accommodate increased loads, the shear capacity of the member must be checked to ensure that it is still sufficient for the new loading. For non-structural FRP applications, applicability checks may not be required.

40.20.4 Materials

A typical FRP system consists of a primer, fibers, resin, bonding material (either additional resin or an adhesive), and a protective coating. FRP is specified in terms of the types of fiber and resin, the number of layers, the fiber orientation and the geometry. FRP is sold as a system and all materials used should be from the same system.

40.20.4.1 Fibers

The most common types of fiber used for bridge repairs are glass and carbon. Glass fibers are not as stiff or as strong as carbon, but they are much less expensive. Unless there is reason to do otherwise, it is recommended that glass fibers be used for corrosion protection and spall control. Carbon fibers should be used for strengthening and crack control.

Carbon fibers cannot be used where the FRP comes into contact with steel out of concerns for galvanic corrosion due to the highly conductive nature of carbon fibers. For applications where galvanic corrosion is a concern, glass fibers may be used, even in structural applications.

Often, FRP is requested by the region to provide column confinement. The engineer must determine if the requested confinement is true confinement where the FRP puts the column into triaxial compression to increase the capacity and ductility, or if the FRP is confining a patch from spalling off. In the case of true confinement (which is very rare in Wisconsin), carbon fibers should be used and the repair requires structural design. For spall control, glass fibers are acceptable and the repair is considered non-structural.

40.20.4.2 Coatings

After the FRP has been installed and fully cured, a protective coating is applied to the entire system. A protective coating is needed to protect against ultraviolet degradation and can also provide resistance to abrasion, wear, and chemicals. In situations where the FRP is submerged in water, inert protective coatings can help prevent compounds in the FRP from leaching into the water, mitigating environmental impacts.

Protective coatings can be made from different materials depending on the desired coating characteristics. Common coating types include vinyl ester, urethane, epoxy, cementitious, and acrylic. Acrylic coatings are generally the least expensive and easiest to apply, though they may also be less durable. If no coating type is specified, it is likely that the manufacturer will provide an acrylic coating.

For shorter term repairs, acrylic coatings are sufficient, but longer repairs should consider other coating types such as epoxy. Any coating used must be compatible with the FRP system selected by the contractor.

40.20.4.3 Anchors

The bond between the FRP and the concrete is the most critical component of an FRP installation and debonding is the most common FRP failure mode. Certain FRP configurations use anchors to increase the attachment of the FRP and attempt to delay or prevent debonding. These anchors can consist of near surface mounted bars, fiber anchors, additional FRP strips, or mechanical anchors such as bolts. It is permitted to use additional U-wrap strips to anchor flexural FRP, but the use of additional longitudinal strips to anchor shear FRP is prohibited. The use of additional U-wrap strips for flexural anchorage is required in some instances.

Because neither design guide requires anchorage or provides information as to what constitutes anchorage, it is left to the discretion of the designer to determine if anchorage should be used and in what quantities. The use of anchors is highly encouraged, particularly for shear applications and in situations where there is increased potential for debonding such as reentrant corners.

Specifying anchors will add cost to the repair, but these costs may be offset by increased capacity accorded to anchored systems in shear. The additional costs can also be justified if debonding is a concern. If the designer chooses to use anchors, anchors should be shown on plans, but the design of the anchors is left to the manufacturer.

40.20.5 Flexure

Flexural FRP is applied along the tension face of the member, where it acts as additional tension reinforcement. The fibers should be oriented along the length of the member.

40.20.5.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For flexure, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

40.20.5.2 Composite Action

Composite action of the deck slab can be considered when designing flexural FRP repairs for girders, provided that the deck was designed to be composite. If composite action is

considered, composite section properties must be computed. These properties should be substituted into the design equations presented in this section. Accounting for composite action will increase the capacity provided by the FRP.

40.20.5.3 Pre-Existing Substrate Strain

Unless all loads are removed from the member receiving FRP (including self-weight), there will be strain present in the concrete when the FRP is applied. This initial or pre-existing substrate strain ε_{bi} is computed through elastic analysis. All loads supported by the member during FRP installation should be considered and cracked section properties should be considered if necessary.

40.20.5.4 Deflection and Crack Control

Conduct standard LRFD serviceability checks for deflection and crack control while accounting for the contribution of the FRP. Because both the FRP and the concrete will be in the elastic zone at service levels, standard elastic analysis can be used to determine stresses and strains. Transformed section analysis can be used to transform the FRP into an equivalent area of concrete for the purposes of analysis. The condition of the member determines if the cracked or uncracked section properties should be used in computations.

40.20.6 Shear

In shear repair/retrofitting applications, the FRP acts essentially as external stirrups. The FRP wrap is applied with the fibers running transverse to the member.

40.20.6.1 Pre-Design Checks

If the design of the FRP will be provided by the contractor or their consultant, the engineer must still perform certain checks before specifying FRP in the plans. For shear, the engineer must check that the structure has sufficient capacity to carry service loads without additional capacity from the FRP, as discussed in 40.20.3.

Additionally, the engineer must ensure that the amount of FRP capacity required does not exceed the maximum allowable shear reinforcement. <u>It is important to note that the FRP capacity listed on the plans will be a factored capacity</u>, while the maximum allowable shear reinforcement check is for an unfactored capacity. Strength reduction factors must be incorporated to make a proper comparison.

If the FRP capacity is close to the maximum allowed, the designer must take care to ensure that a design is feasible. The capacity provided by FRP depends on the number of FRP layers, with each additional layer providing a discrete increase in capacity. There may be a situation where n layers does not provide enough capacity, but n+1 layers provides too much capacity and violates the maximum allowable shear reinforcement criteria. Changes in spacing of the wraps may help decrease the capacity provided by the FRP.

Example problems in shear can be found in the appendices of NCHRP Report 655 (16) and potential shear wrapping configurations can be found in NCHRP Report 678 (17).

40.21 References

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3. *Durable Bridge Decks* by D. G. Manning and J. Ryell, Ontario Ministry Transportation and Communications, April, 1976.

4. *Durability of Concrete Bridge Decks, NCHRP Report 57*, May, 1979.

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7. *Latex Modified Concrete Bridge Deck Overlays - Field Performance Analysis* by A. G. Bisharu, Report No. FHWA/OH/79/004, October, 1979.

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9. *The Effect of Moving Traffic on Fresh Concrete During Bridge Deck Widening* by H. L. Furr and F. H. Fouad, Paper Presented 61 Annual TRB Meeting, January, 1982.

10. *Control of Cracking in Concrete Structures by ACI Committee 224*, Concrete International, October, 1980.

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15. ACI. 2008. *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.* Farmington Hills MI: American Concrete Institute, 2008. ACI 440.2R-08.

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19. Tabatabai, H., K. Sobolev, A. Ghorbanpoor, A. Nabizadeh, C-W. Lee, M. Lind. 2016. *Evaluation of Thin Polymer Overlays for Bridge Decks*. Wisconsin Highway Research Program, Madison, WI.

20. Bektas, B., Albughdadi, A., Freeseman, K., Bazargani, B. 2020. *Protocols for Concrete Bridge Deck Protections and Treatments*. Wisconsin Highway Research Program, Madison, WI.

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The first step is to perform a rating analysis to determine inventory and operating ratings. Based on the results of the rating analysis, a posting analysis should be performed when:

- The inventory rating factor is less than or equal to 1.0 (HS-20) – Emergency Vehicles (EVs) only, see Figure 45.10-5; or

- The operating rating factor is less than or equal to 1.3 (HS-26) – Specialized Hauling Vehicles (SHVs) only, see Figure 45.10-2; or

- The operating rating factor is less than or equal to 1.0 (HS-20) for all other posting vehicles.

An emergency vehicle analysis is performed to determine whether a bridge can safely carry emergency vehicles, which may exceed legal weight limits in place for other vehicles. A posting analysis is performed to determine whether a bridge can safely carry other legal-weight traffic. Both analyses are performed at the operating level. See 45.10 for more information.

Permit analysis is used to determine whether or not over legal-weight vehicles may travel across a bridge. See 45.11 for more information on over-weight vehicle permitting.

A flow chart outlining this approach is shown in Figure 45.3-2. The procedures are structured to be performed in a sequential manner, as needed.
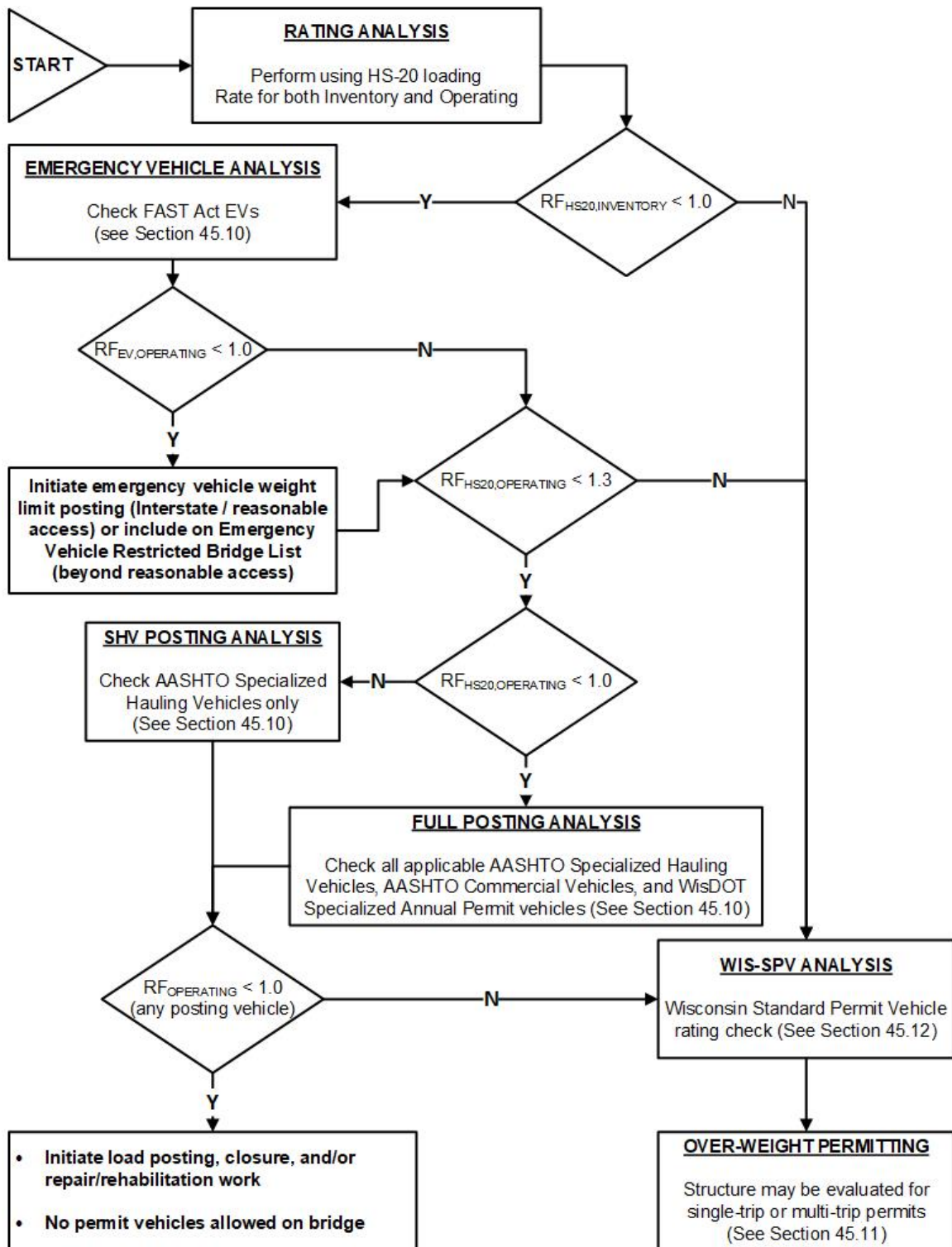
45.3.8.1 Load Factors for Load Factor Rating

See Table 45.3-5 for load factors to be used when rating with the LFR method. The nominal capacity, C, is the same regardless of the rating level desired.

For emergency vehicles, alternate live load factors determined in accordance with NCHRP Project 20-07 / Task 410 may be used. If alternate live load factors are used, this shall be noted in the Load Rating Summary Form, along with assumptions of one-way ADTT and emergency vehicle crossings per day.

LFR Load Factors		
Rating Level	A_1	A_2
Inventory	1.3	2.17
Operating	1.3	1.3

Table 45.3-5
LFR Load Factors

Figure 45.3-2
Load Factor Rating and Allowable Stress Rating Flow Chart

45.3.8.2 Live Loads for Load Factor Rating

Similar to LRFR, there are three potential checks to be made in LFR that are detailed in the flow chart shown in Figure 45.3-2.

- For purposes of calculating the inventory and operating rating of the structure, the live load to be used should be the HS20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3.

- The live load(s) to be used for analysis are a combination of AASHTO-prescribed vehicles, Wisconsin-specific vehicles, and FAST Act emergency vehicles. For more information on load posting analysis, refer to 45.10.2.

- For conducting the Wisconsin Standard Permit Vehicle analysis, use the loading shown in Figure 45.12-1.

45.3.8.3 Load Distribution for Load Factor Rating

In general, distribution factors should be calculated based on the guidance of the *AASHTO Standard Design Specifications, 17th Edition*.

See 45.5.1.2 for specific direction on the placement of live loads for rating and posting.

Dead loads shall be distributed as described in 17.2.7 for concrete slab superstructures and 17.2.8 for concrete deck on girder superstructures.

45.3.9 Allowable Stress Rating (ASR)

The basic rating equation can be found in **MBE [Equation 6B.4.1-1]** and is:

$$RF = \frac{C - D}{L(1 + I)}$$

Where:

RF	=	Rating factor for the live load carrying capacity
C	=	Capacity of the member
D	=	Dead load effect on the member
L	=	Live load effect on the member
I	=	Impact factor to be used with the live load effect

Unlike LRFR, allowable stress rating does not have three prescribed levels of rating analysis. However, in practice, the process is similar for both LRFR and ASR.

The first step is to perform a rating analysis to determine inventory and operating ratings. Based on the results of the rating analysis, a posting analysis should be performed when:

- The inventory rating factor is less than or equal to 1.0 (HS-20) – Emergency Vehicles (EVs) only, see Figure 45.10-5; or

- The operating rating factor is less than or equal to 1.3 (HS-26) – Specialized Hauling Vehicles (SHVs) only, see Figure 45.10-2; or

- The operating rating factor is less than or equal to 1.0 (HS-20) for all other posting vehicles.

An emergency vehicle analysis is performed to determine whether a bridge can safely carry emergency vehicles, which may exceed legal weight limits in place for other vehicles. A posting analysis is performed to determine whether a bridge can safely carry other legal-weight traffic. Both analyses are performed at the operating level. See 45.10 for more information.

Permit analysis is used to determine whether or not over legal-weight vehicles may travel across a bridge. See 45.11 for more information on over-weight vehicle permitting.

A flow chart outlining this approach is shown in Figure 45.3-2. The procedures are structured to be performed in a sequential manner, as needed.

45.3.9.1 Stress Limits for Allowable Stress Rating

The inventory and operating stress limits used in ASR vary by material. See **MBE [6B]** for more information.

45.3.9.2 Live Loads for Allowable Stress Rating

Similar to LRFR and LFR, there are three potential checks to be made in ASR.

- For purposes of calculating the inventory and operating rating of the structure, the live load to be used should be the HS-20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3.

- The live load(s) to be used for analysis are a combination of AASHTO-prescribed vehicles, Wisconsin-specific vehicles, and FAST Act emergency vehicles. For more information on load posting analysis, refer to 45.10.2.

- For conducting the Wisconsin Standard Permit Vehicle analysis, use the loading shown in Figure 45.12-1.

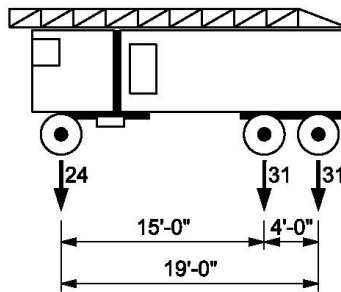### 45.3.9.3 Load Distribution for Allowable Stress Rating

In general, distribution factors should be calculated based on the guidance of the *AASHTO Standard Design Specifications, 17th Edition*.

Indicated concentrations
are axle loads in kips.
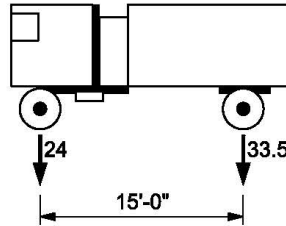
EV2 Unit Weight = 57.5 Kips (28.75 tons)

EV3 Unit Weight = 86 Kips (43 tons)

Figure 45.10-5
Emergency Vehicle Load Models

45.10.3 Load Posting Analysis

All posting vehicles shall be analyzed at the operating level. A load posting analysis is required when the calculated rating factor at operating level for a bridge is:

- Less than 1.0 for HL-93 loading using LRFR methodology.

- Less than 1.0 for HS-20 loading using LFR/ASR methodology; or

- Less than or equal to 1.3 for LFR/ASR methodology (SHV analysis only)

A load posting analysis is very similar to a load rating analysis, except the posting live loads noted in 45.10.2 are used instead of typical LFR or LRFR live loading.

If the calculated rating factor at operating is less than 1.0 for a given load posting vehicle, then the bridge shall be posted, with the exception of the Wis-SPV. For State Trunk Highway Bridges, current WisDOT policy is to post structures with a Wisconsin Standard Permit Vehicle (Wis-SPV) rating of 120 kips or less. If the RF ≥ 1.0 for a given vehicle at the operating level, then a posting is not required for that particular vehicle.

A bridge is posted for the lowest restricted weight limit of any of the standard posting vehicles. To calculate the capacity, in tons, on a bridge for a given posting vehicle utilizing LFR, multiply the rating factor by the gross vehicle weight in tons. To calculate the posting load for a bridge analyzed with LRFR, refer to 45.10.3.2.

Posting or weight limit analysis for emergency vehicles occurs separately; it is required when the calculated rating factor at inventory level for a bridge is:

- Less than 0.9 for HL-93 loading using LRFR methodology; or

- Less than 1.0 for HS-20 loading using LFR/ASR methodology.

If the calculated rating factor at operating rating is less than 1.0 for a given emergency vehicle, then the bridge shall have an emergency vehicle-specific weight limit restriction, as follows:

- If RF_{EV2} < 1.0 and RF_{EV3} < 1.0
    - Single Axle = Minimum (RF_{EV2} x 16.75 tons, RF_{EV3} x 31 tons)
    - Tandem = Minimum (RF_{EV2} x 28.75 tons, RF_{EV3} x 31 tons)
    - Gross = Minimum (RF_{EV2} x 28.75 tons, RF_{EV3} x 43 tons)
- If only RF_{EV2} < 1.0
    - Single Axle = RF_{EV2} x 16.75 tons
    - Tandem = RF_{EV2} x 28.75 tons
    - Gross = RF_{EV2} x 28.75 tons
- If only RF_{EV3} < 1.0
    - Single Axle = Minimum (16 tons, RF_{EV3} x 31 tons)
    - Tandem = RF_{EV3} x 31 tons
    - Gross = RF_{EV3} x 43 tons

Sign postings may or may not be required for emergency vehicles, depending on their location. Refer to 45.10.4.

45.10.3.1 Limit States for Load Posting Analysis

For LFR methodology, load posting analysis should consider strength-based limit states only.

For LRFR methodology, load posting analysis should consider strength-based limit states, but also some service-based limit states, per Table 45.3-1.

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E45-4 Steel Girder Rating Example - LRFR

This example shows rating calculations conforming to the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* as supplemented by the *WisDOT Bridge Manual (July 2008)*. This example will rate the design example E24-1 contained in the WisDOT Bridge Manual. **(Note: Example has not been updated for example E24-1 January 2016 updates)**

E45-4.1 Preliminary Data

An interior plate girder will be rated for this example. The girder was designed to be composite throughout. There is no overburden on the structure. In addition, inspection reports reveal no loss of section to any of the main load carrying members.
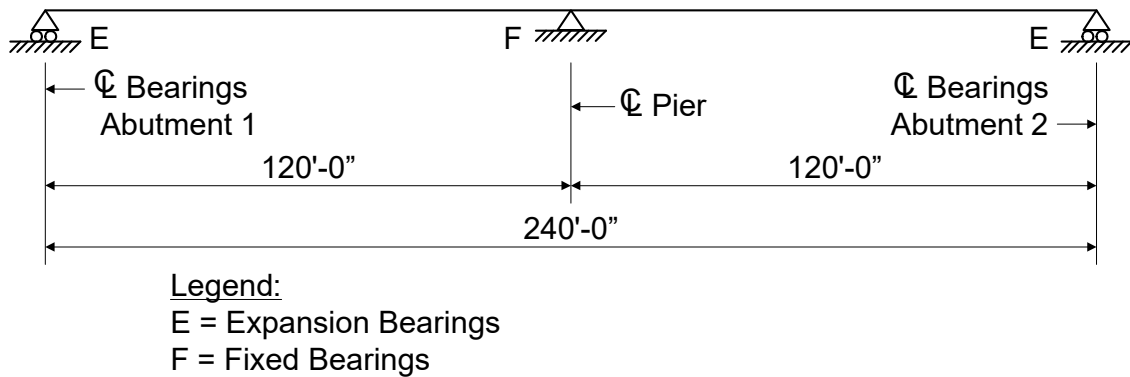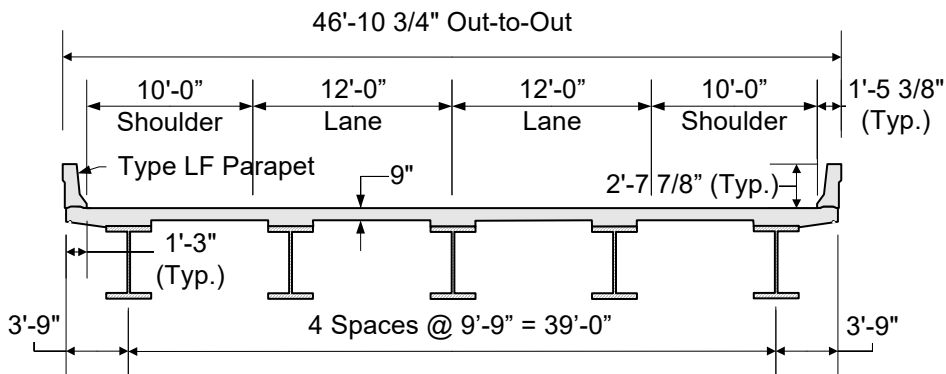

Legend:
E = Expansion Bearings
F = Fixed Bearings

Figure E45-4.1-1
Span Configuration

Figure E45-4.1-2
Superstructure Cross Section

Dead Load Shears (Kips)

Dead Load Component	Location in Span 1										
	0.0L	**0.1L**	**0.2L**	**0.3L**	**0.4L**	**0.5L**	**0.6L**	**0.7L**	**0.8L**	**0.9L**	**1.0L**
Steel girder	7.0	5.0	2.9	0.9	-1.1	-3.1	-5.2	-7.2	-9.8	-12.9	-17.0
Concrete deck & haunches	46.9	33.2	19.5	5.8	-7.9	-21.6	-35.3	-49.0	-62.6	-76.1	-89.5
Miscellaneous Steel Weight	1.2	0.9	0.5	0.2	-0.2	-0.6	-0.9	-1.3	-1.7	-2.0	-2.4
Concrete parapets	6.6	4.7	2.9	1.0	-0.9	-2.7	-4.6	-6.5	-8.3	-10.2	-12.0
Future wearing surface	7.5	5.4	3.2	1.1	-1.0	-3.1	-5.2	-7.3	-9.4	-11.6	-13.7

Table 45E-4.3-3
Dead Load Shears

E45-4.4 Compute Live Load Distribution Factors for Interior Girder

The live load distribution factors for an interior girder are computed as follows **LRFD [4.6.2.2.2]**:

First, the longitudinal stiffness parameter, K_g, must be computed **LRFD [4.6.2.2.1]**:

$$K_g := n \cdot \left(I + A \cdot e_g^2 \right)$$

Where:

I	= Moment of inertia of beam (in^4)
A	= Area of stringer, beam, or girder (in^2)
e_g	= Distance between the centers of gravity of the basic beam and deck (in)

Longitudinal Stiffness Parameter, K_g				
	Region A (Pos. Mom.)	**Region B (Intermediate)**	**Region C (At Pier)**	**Weighted Average ***
Length (Feet)	84	20	16	
n	8	8	8	
I (Inches^4)	23,605.3	34,639.8	65,426.6	
A (Inches^2)	49.750	63.750	100.500	
e_g (Inches)	35.978	35.777	36.032	
K_g (Inches^4)	704,020	929,915	1,567,250	856,767

Table E45-4.4-1
Longitudinal Stiffness Parameter

After the longitudinal stiffness parameter is computed, **LRFD [Table 4.6.2.2.1-1]** is used to find the letter corresponding with the superstructure cross section. The letter corresponding with the superstructure cross section in this design example is "a."

If the superstructure cross section does not correspond with any of the cross sections illustrated in **LRFD [Table 4.6.2.2.1-1]**, then the bridge should be analyzed as presented in **LRFD [4.6.3]**.

Based on cross section "a", **LRFD [Table 4.6.2.2.2b-1 & Table 4.6.2.2.3a-1]** are used to compute the distribution factors for moment and shear, respectively.

For the 0.4L point:

$K_g = 856766.65$ in^4

$L := 120$ ft

For one design lane loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [Table 4.6.2.2.2b-1]**:

$$g_{m1} := 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0L \cdot t_s{}^3}\right)^{0.1}$$

$$\boxed{g_{m1} = 0.466}$$ lanes

For two or more design lanes loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [Table 4.6.2.2.2b-1]**:

$$g_{m2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0 \cdot L \cdot t_s{}^3}\right)^{0.1}$$

$$\boxed{g_{m2} = 0.688}$$ lanes

The live load distribution factors for shear for an interior girder are computed in a similar manner. The range of applicability is similar to that for moment **LRFD [Table 4.6.2.2.3a-1]**.

For one design lane loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{v1} := 0.36 + \frac{S}{25.0}$$ $$\boxed{g_{v1} = 0.750}$$ lanes

For two or more design lanes loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{v2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$ $$\boxed{g_{v2} = 0.935}$$ lanes

Since this bridge has no skew, the skew correction factor does not need to be considered for this design example **LRFD [4.6.2.2.2e & 4.6.2.2.3c]**.

HL-93 Live Load Effects (for Interior Beams)

Live Load Effect	Location in Span 1										
	0.0L	**0.1L**	**0.2L**	**0.3L**	**0.4L**	**0.5L**	**0.6L**	**0.7L**	**0.8L**	**0.9L**	**1.0L**
Maximum positive moment (K-ft)	0.0	848.1	1435.4	1780.1	1916.6	1859.0	1626.9	1225.6	699.7	263.6	0.0
Maximum negative moment (K-ft)	0.0	-134.3	-268.7	-403.0	-537.4	-671.7	-806.0	-940.4	-1087.0	-1591.6	-2414.2
Maximum positive shear (kips)	111.1	92.9	76.0	60.4	46.4	34.0	23.3	14.5	7.6	3.0	0.0
Maximum negative shear (kips)	-15.2	-15.7	-21.9	-35.0	-49.2	-63.6	-78.1	-92.3	-106.1	-119.3	-132.0

Table 45E-4.4-2
Live Load Effects

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E45-8 Steel Girder Rating Example - LFR

Reference E45-4 for bridge data. For LFR, the Bureau of Structures rates structures for the Design Load (HS20) and for Permit Vehicle loads. The rating below analyzes an interior girder only, which typically governs.

E45-8.1 Preliminary Data

$N_{spans} := 2$		Number of spans	
$L := 120$	ft	span length	
$N_b := 5$		number of girders	
$S := 9.75$	ft	girder spacing	
$L_b := 240$	in	cross-frame spacing	
$F_{yw} := 50$	ksi	web yield strength	
$F_{yf} := 50$	ksi	flange yield strength	
$f'_c := 4.0$	ksi	concrete 28-day compressive strength	
$f_y := 60$	ksi	reinforcement strength	
$E_s := 29000$	ksi	modulus of elasticity	
$t_{deck} := 9.0$	in	total deck thickness	
$t_s := 8.5$	in	effective deck thickness when 1/2" wearing surface is removed from total deck thickness	
$w_s := 0.490$	kcf	steel density **Std [3.3.6]**	
$w_c := 0.150$	kcf	concrete density **Std [3.3.6]**	
$w_{misc} := 0.030$	kip/ft	additional miscellaneous dead load (per girder) per 17.2.4.1	
$w_{par} := 0.387$	kip/ft	parapet weight (each)	
$w_{deck} := 46.5$	ft	deck width	
$d_{haunch} := 3.5$	in	haunch depth (from top of web for design) (for construction, the haunch is measured from the top of the top flange)	

Therefore

$$RF_{vf1\_inv} := RF_{mvf1\_inv} \qquad \boxed{RF_{vf1\_inv} = 1.39}$$

$$RF_{mf1\_inv} := RF_{mvf1\_inv} \qquad \boxed{RF_{mf1\_inv} = 1.39}$$

$$RF_{vf1\_op} := RF_{mvf1\_op} \qquad \boxed{RF_{vf1\_op} = 2.32}$$

$$RF_{mf1\_op} := RF_{mvf1\_op} \qquad \boxed{RF_{mf1\_op} = 2.32}$$

Step 5 - If the controlling RF is different than the initial controlling RF, repeat Steps 2-4 (using the final controlling RF as the initial controlling RF):

$$RF_{mv2\_inv} := \min\left(RF_{vf1\_inv}, RF_{mf1\_inv}\right) \qquad RF_{mv2\_inv} = 1.39$$

$$V_2 := 1.3 \cdot V_D + RF_{mv2\_inv} \cdot 2.17 \cdot V_L \qquad \boxed{V_2 = 400.4} \qquad \text{kips}$$

$$V_2 > 0.6 V_n$$

$$f_2 := 1.3 \cdot f_D + RF_{mv2\_inv} \cdot 2.17 \cdot f_L \qquad \boxed{f_2 = 44.36} \qquad \text{ksi}$$

$$M_2 > 0.75 M_{n\_neg}$$

CASE D applies again, so the calculation does not need to be repeated.

$$RF_{mvf\_inv} := RF_{mf1\_inv} \qquad \boxed{RF_{mvf\_inv} = 1.39}$$

$$RF_{mvf\_op} := RF_{mf1\_op} \qquad \boxed{RF_{mvf\_op} = 2.32}$$

Since RF>1.30 @ operating for all checks, posting vehicle checks are not required for this example.

E45-8.12 - Permit Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.12).

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming full dynamic load allowance is utilized. Future wearing surface shall not be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW. Future wearing surface shall be included in the check.

E45-8.12.1 - Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

The values from this analysis are used for performing the Wis-SPV design check per 45.12

Load Distribution Factors

Single Lane Interior DF $\boxed{DF_s = 1.39}$

Wis-SPV Moments and Shears from LL analysis software, with impact and distribution factors included:

$M_{LL\_0.4L} := 2393.45$ kip-ft

$M_{LL\_1.0L} := 1836.47$ kip-ft

$V_{LL\_1.0L} := 132.47$ kips

The DL moments and shears with wearing surface included are:

$M_{DL\_0.4L} := M_{girder} + M_{deck} + M_{misc} + M_{DC2} + M_{DW}$

$\boxed{M_{DL\_0.4L} = 1379}$ kip-ft

$M_{DL\_1.0L} := -\left(M_{girder\_neg} + M_{deck\_neg} + M_{misc\_neg} + M_{DC2\_neg} + M_{DW\_neg}\right)$

$\boxed{M_{DL\_1.0L} = 3787}$ kip-ft

$V_{DL\_1.0L} := -\left(V_{girder} + V_{deck} + V_{misc} + V_{DC2} + V_{DW}\right)$

$\boxed{V_{DL\_1.0L} = 134.7}$ kips

In continuous spans with noncompact noncomposite or composite negative-moment pier sections, the maximum bending strength, M_n, of the composite positive-moment sections shall be taken as either the moment capacity at first yield or as:

$M_n := M_y + A \cdot \left(M_{u\_pier} - M_{s\_pier}\right)$ **Std [Eq. 10-129d]**

Where:

M_y	=	the moment capacity at first yield of the compact positive moment section
$(M_{u\_pier} - M_{s\_pier})$ =		moment capacity of the noncompact section at the pier from [10.48.2] or [10.48.4] minus the elastic moment at the pier for the loading producing maximum positive bending in the span.
A	=	distance from end support to the location of maximum positive moment divided by the span length for end spans.