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2.1 Organizational Charts

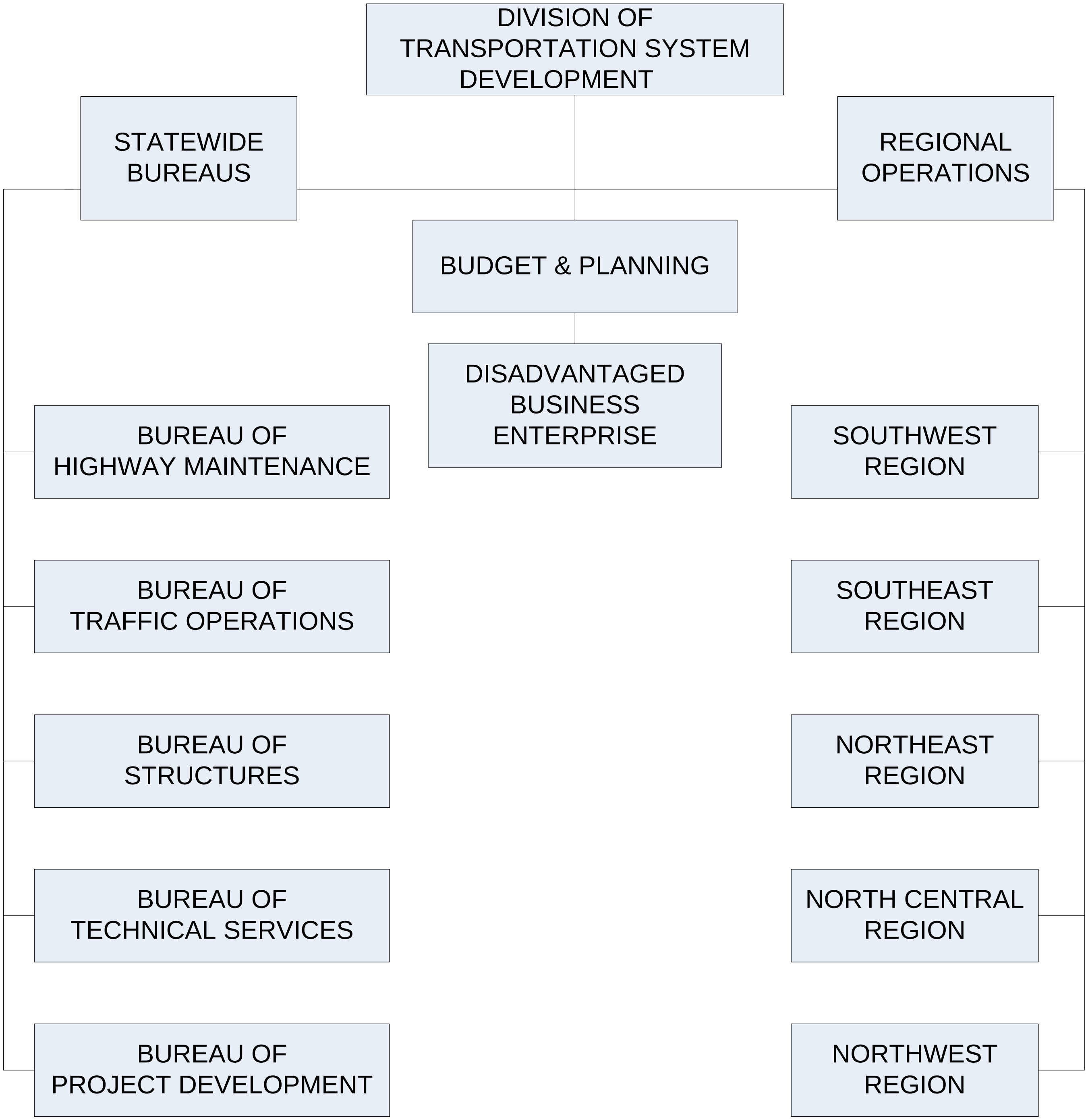


Figure 2.1-1
Division of Transportation System Development

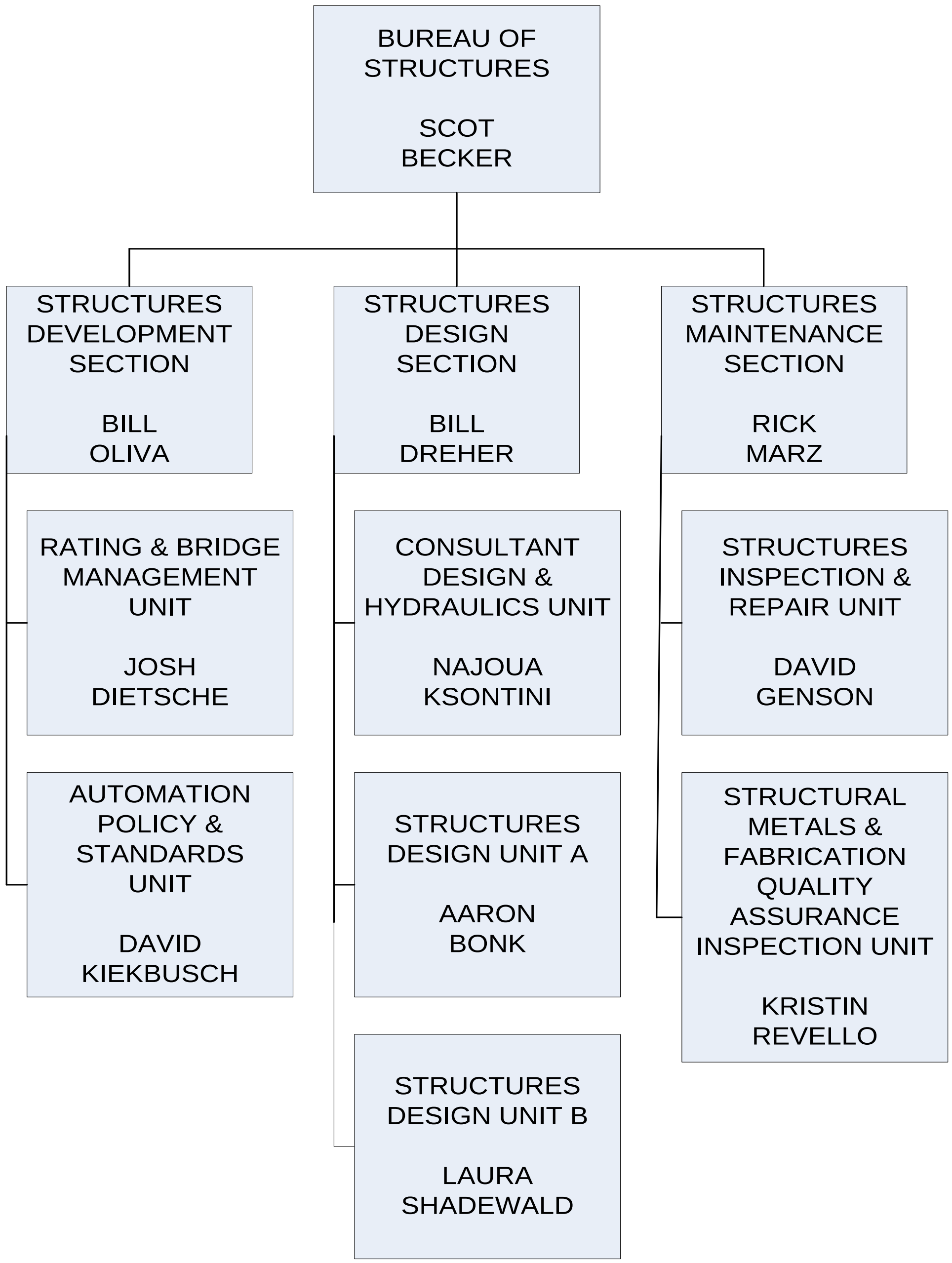
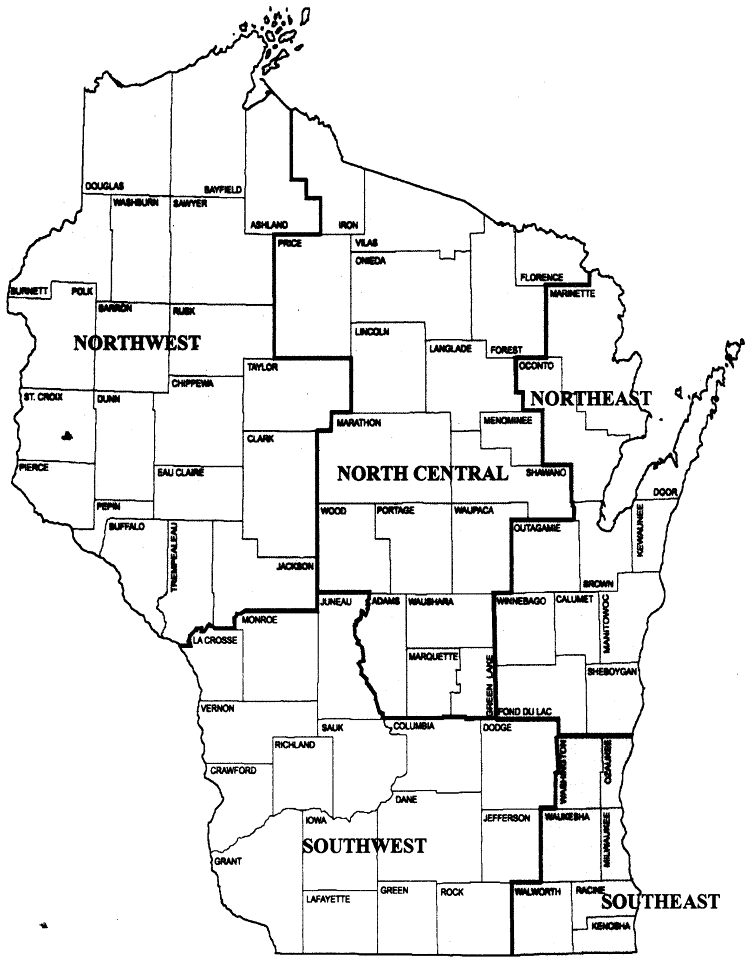


Figure 2.1-2
Bureau of Structures



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

Figure 2.1-3
Region Map

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3.1 Specifications and Standards

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

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

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

3.2 Geometrics and Loading

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

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

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

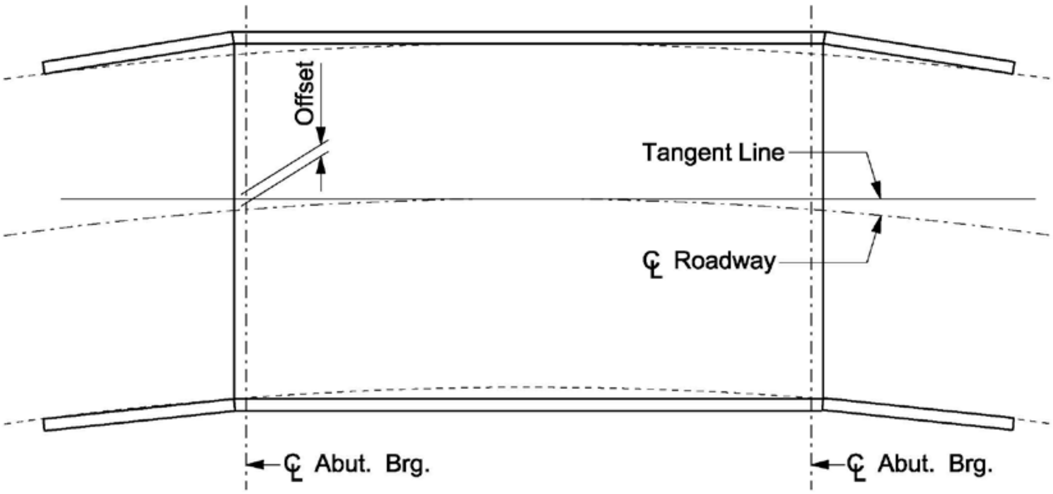
Highway bridge design live loadings follow the AASHTO LRFD Design specifications using HL93. Chapter 17 provides more detail on applying this load for design. WisDOT requires a specific vehicle design check using the Wis-SPV (Standard Permit Vehicle) which can be found in Chapter 45.

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

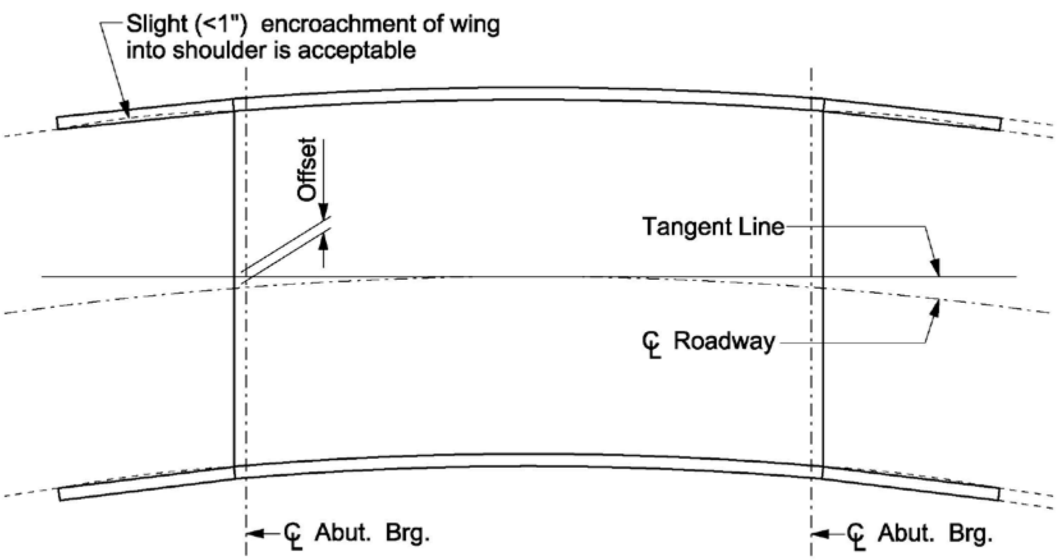
All new bridges constructed in the State of Wisconsin are designed for the clearances shown in FDM Procedure 11-35-1, Attachment 1.8. FDM Procedure 11-35-1, Attachment 1.9 covers the cases described in that section as well as bridge widenings. Wires and cables over highways are designed for clearances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

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

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



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



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

Figure 3.2-1
Bridge Layout on Horizontal Curves

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4.1 Introduction

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

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

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

4.3 Primary Features

Superstructure Type and Shape

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

- Larger overhangs to create better shadow lines.
- Horizontal recess on the backside of the parapet, which could be stained or left as plain concrete.
- Eliminate or minimize pedestals along the parapet. Such pedestals tend to break up the horizontal flow and make the superstructure appear top heavy. Pedestals, if desired, are better left on the wings to delineate the beginning or end of the bridge or to frame the bridge when viewed from below. If used on the superstructure, keep the pedestal size smaller and space apart far enough to avoid a top heavy appearance. See Chapter 30 – Railings for further guidance.
- Minimize vertical or patterned elements on the backside of the parapet as such elements tend to break up the horizontal flow. Rock form liner has become an overused aesthetic enhancement for the backside of parapets, as its use oftentimes does not fit the surroundings. See Chapter 30 – Railings for further guidance.

Abutment Type and Shape

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

Pier Type and Shape

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

Grade and/or Skew

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

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

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

4.5 Aesthetics Process

A number of parties can be responsible for the appearance of a structure, as well as the project as a whole. The structural design engineer should be instrumental in leading the aesthetic design process, a process that may include the Region, the Bureau of Structures, the public and aesthetic advisors (architects, landscape architects, urban planners, artists, etc).

Public input comes in a variety of ways. Advisory groups, special interest groups and general public information meetings are all ways to receive public input and are part of the CSS (Community Sensitive Solutions) process.

The structural design engineer needs to be involved early in the aesthetic decision making process. BOS should have early representation on projects with considerable aesthetic concerns.

WisDOT policy item:

The budget bill passed in July, 2015 reduced State CSS funding to zero. Very low cost aesthetic enhancements through appropriate shape and geometric relief are allowed. See 4.3 for discussion on primary features such as shape. Geometric relief is defined as:

- Rustications produced by cut (likely) wood (e.g. rustication lines)
- Formliners such as ribbed or broken ribbed
- Formliners that do not replicate other objects (e.g. rocks or cut stone)
- Shapes that do not depict anything pictorially (e.g. animals, flowers, sailboats, etc.)

Items considered CSS (not state funded)*:

- Stain
- Formliner, other than the geometric formliner defined above
- Pedestrian railing or fencing other than that shown on Standards for Combination Railing Type '3T' (galvanized, only) or Chain Link Fence Details (galvanized, only)
- Ornamentation, including city symbols, street names, etc.
- Non-standard lighting and sign supports
- Structure shapes that are not as defined in 4.3

* CSS items also require a State Municipal Agreement that makes local municipalities responsible for future maintenance and all associated costs.

4.6 Levels of Aesthetics

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

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

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

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

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

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

Note: The above text was left in this chapter, but will likely be modified or removed in future editions of this Manual. See 4.9 for current policy regarding CSS and levels of aesthetics.

4.9 Non-CSS Aesthetic Concepts

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

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

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

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

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

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

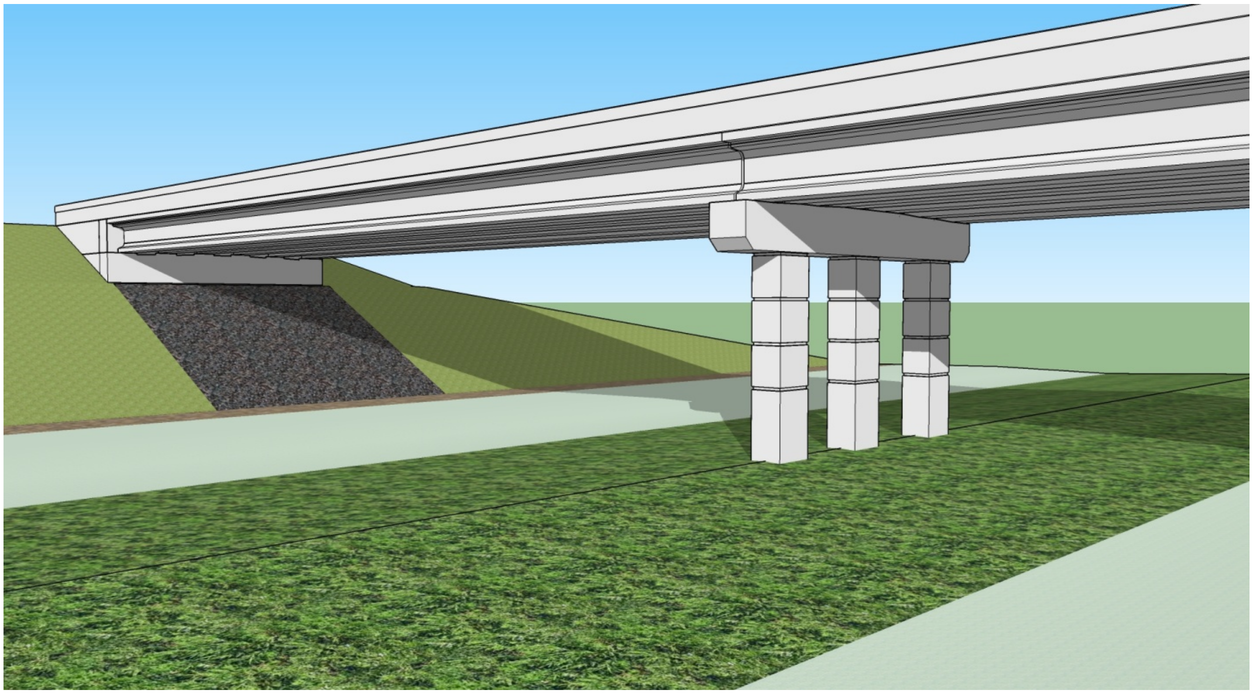

Figure 4.9-1
Aesthetic Concept Type I

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

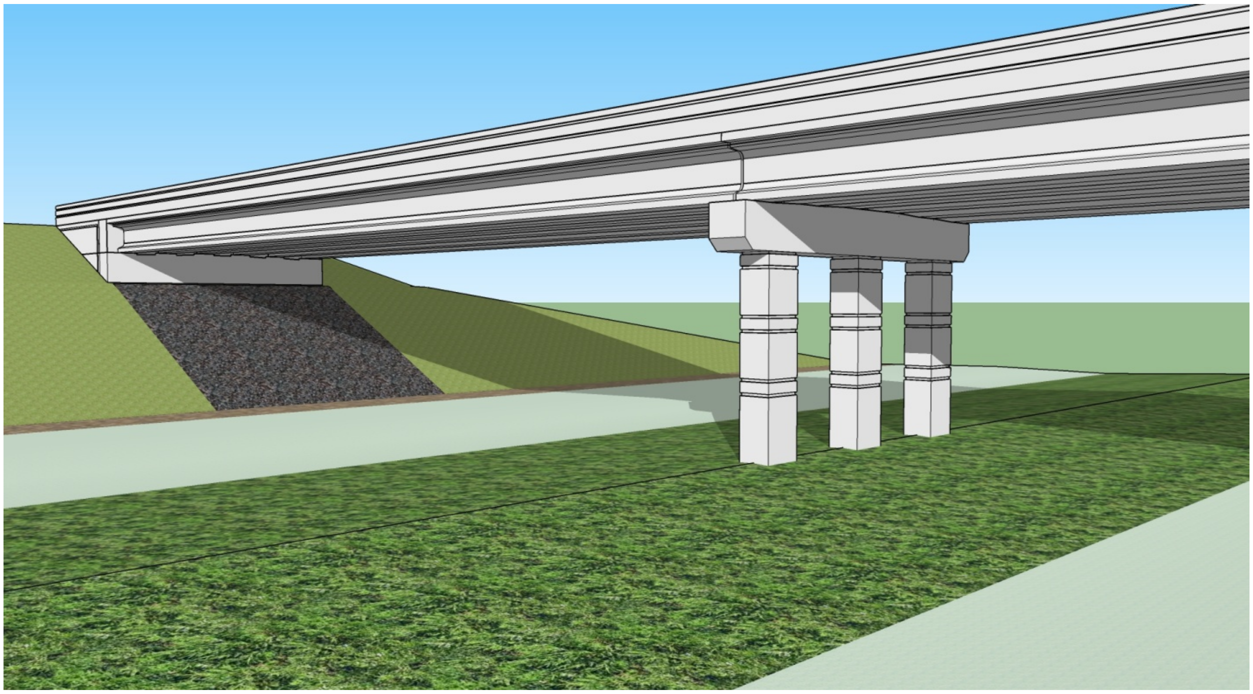

Figure 4.9-3
Aesthetic Concept Type II

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

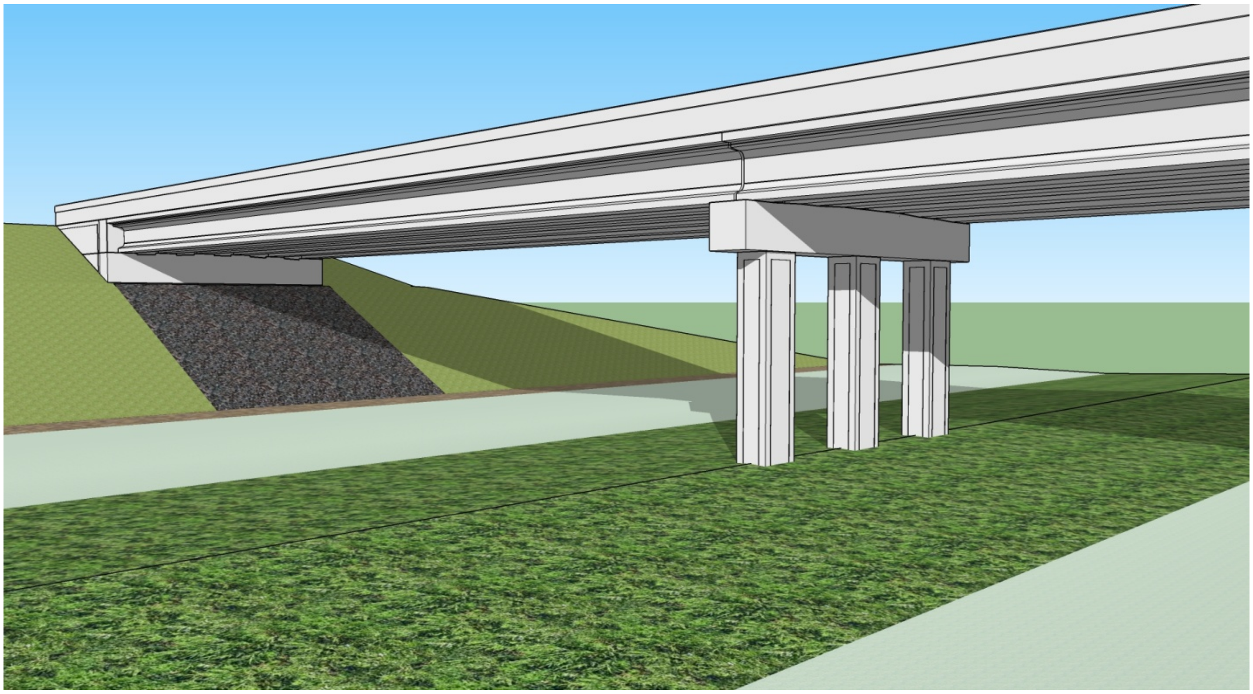

Figure 4.9-2
Aesthetic Concept Type III

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

4.10 References

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

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

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5.1 Factors Governing Bridge Costs

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

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

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

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

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

5.4.5 2014 Year End Structure Costs

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

Table 5.4-28
Stream Crossing Structures

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

Table 5.4-29
Grade Separation Structures

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

Table 5.4-30
Box Culverts

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

Table 5.4-31
Retaining Walls

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

Table 5.4-32
Noise Walls

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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

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

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

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

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

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

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

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

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

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

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

6.4.9 Piling Test Treated Timber (Structure)

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

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

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

Record the type, bid quantity in lineal feet. Railing length should be actual length of rail along surface it is attached to (not horizontal length shown on plans).

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

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

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

Record this quantity to the nearest 5 cubic yards.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

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

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

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Device (Structure)

Record this quantity in lump sum.

6.4.38 Concrete Masonry Deck Patching

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.

6.4.40 Removing Bearings

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

6.4.41 Ice Hot Weather Concreting

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

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

The need for structures is determined during the Preliminary Site Survey and recorded in the Concept Definition or Work Study Report. On Federal (FHWA) or State Aid Projects completed Structure Survey Reports and plans are submitted to the Bureau of Structures with a copy forwarded to the Regional Office for approval prior to construction. Structure and project numbers are assigned 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 service loads of the structure.
- A means for verifying the completeness, constructability and accuracy of the structure plans.
- Verification that independent checks, reviews and ratings were performed.

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

6.5.1 Approvals, Distribution, and Work Flow

Consultant	Meet with Regional Office and/or local units of government to determine need.
	Prepare Structure Survey Report including recommendation of structure type.
Geotechnical Consultant	Make site investigation and prepare Site Investigation Report.
Consultant	Prepare Preliminary Plan documents including scour computations for spread footings and/or shallow pile foundations. Record scour critical

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Figure 7.1-2
Prefabricated Abutment

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

7.1.4.1.1 Precast Piers

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

7.1.4.1.2 Application

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

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

WisDOT policy item:

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

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

- Piers shall be designed to allow either cast-in-place or precast concrete construction, but with only cast-in-place detailed on the plans. Differences between construction methods shall be limited to pier column connections, beam seats details, and diaphragm details. If the pier configuration is not able to reasonably accommodate interchangeability between the two constructions optional piers may be exempt from the precast option.

- Multi-column piers (3x4 ft rectangular) grade separations over roadways only.

- Fixed piers supporting prestressed concrete girders only.

- Precast elements shall be limited to 90 kips.

- Deep foundations are recommended when multiple pier caps are used. Shallow foundations may be considered if differential settlement is not expected.

- Integral barriers or crashwalls are currently excluded from the precast option.

- Applications where the top of the footing may become submerged are prohibited.

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

Figure 7.1-5
GRS Abutment Layer During Construction

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

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

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

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

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

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

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

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

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

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

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

7.1.4.2.1 Design Standards

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

7.1.4.2.2 Application

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

1. Scour potential at the abutment locations has been evaluated and is within acceptable limits

2. Water velocities are less than 5 ft/s

3. Adequate freeboard is provided (See Bridge Manual Chapter 8.3.1.5)

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

7.1.4.2.3.3 Superstructure

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

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

7.1.4.2.3.4 Approach Integration

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

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

7.1.4.2.3.5 Design Details

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

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

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

7.1.4.2.4 Design Steps

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

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

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

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

4. Calculate Applicable Loads
   - Lateral Pressures and Stresses

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

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

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

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

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

7.1.4.3 Lateral Sliding

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

Figure 7.1-6
Lateral Sliding

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

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

7.1.4.4 ABC Using Self Propelled Modular Transporter (SPMT)

7.1.4.4.1 Introduction

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

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

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

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

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

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

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

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

Figure 7.1-7
Self Propelled Modular Transporters Moving a Bridge

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

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

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

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

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

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

7.1.4.4.2 Application

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

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

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

3. There is a major navigation channel under the bridge

4. The bridge is an emergency replacement

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

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

7. There is a sensitive environmental issue

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

7.1.4.4.3 Special Provision

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

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

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

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

7.1.4.4.4 Roles and Responsibilities

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

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

Table 7.1-2
SPMT Roles and Responsibilities

7.1.4.4.4.1 WisDOT

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

7.1.4.4.4.2 Designer

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

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

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

7.1.4.4.4.3 Contractor

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

The Contractor is responsible for:

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

The Contractor is required to:

1. Provide all required plans, calculations, etc. in accordance with the specifications.

2. Identify, design and implement any required ground improvements in the BSA and TP.

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

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

1. Utilities – Mitigation Concepts

2. Site Conditions – Structural Analysis of Bridge Along TP

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

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

5. Acquiring real estate

7.1.4.4.5 Temporary Supports

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

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

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

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

7.1.4.4.6 Design Considerations

7.1.4.4.6.1 Bridge Staging Area

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

Figure 7.1-8
Example Bridge Staging Area (BSA)

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

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

7.1.4.4.6.2 Travel Path

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

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

7.1.4.4.6.3 Allowable Stresses

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

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

7.1.4.4.6.4 Pick Points

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

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

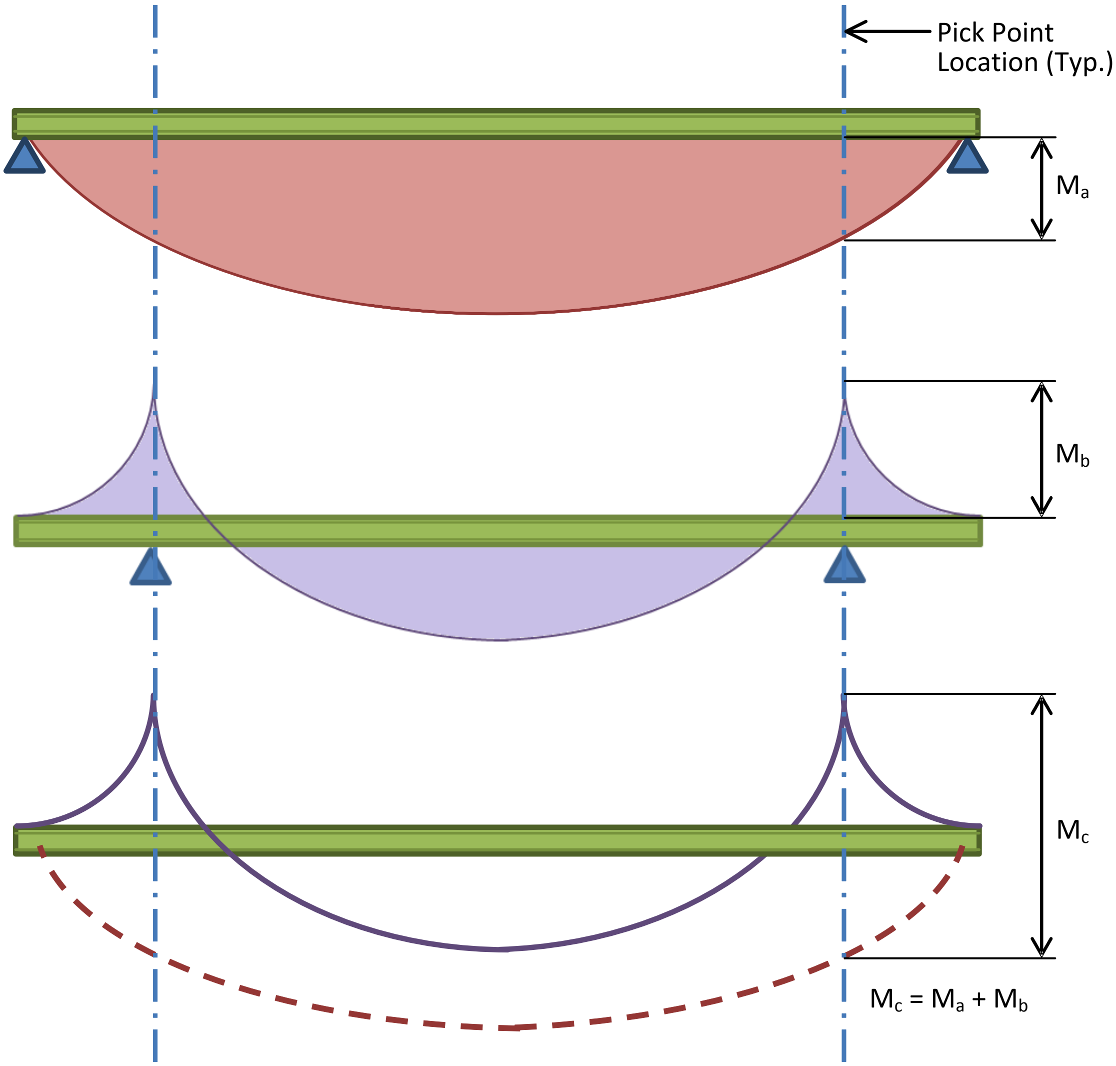

Figure 7.1-9
Support Change Moment Diagram (Illustrating Stress Reversal)

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

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

7.1.4.4.6.5 Deflection and Twist

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

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

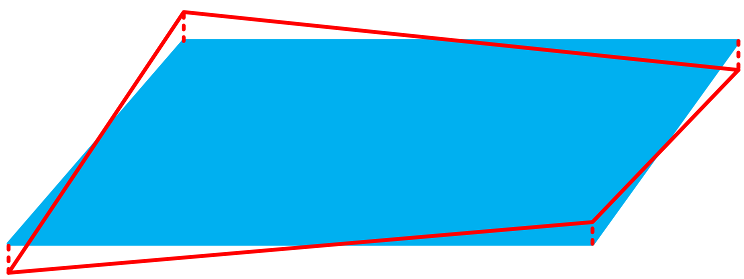

Figure 7.1-10
Bridge Warping Diagram

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

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

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

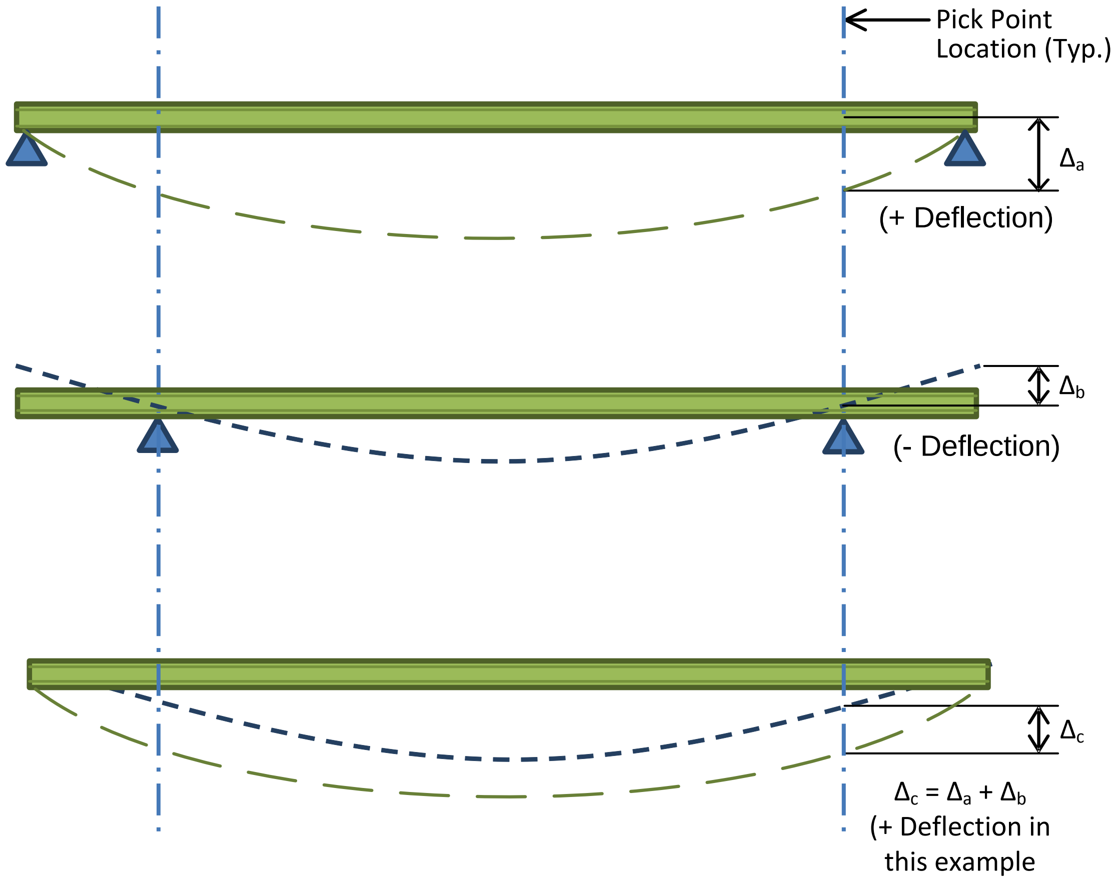

Figure 7.1-11
Support Change Deflection Diagram

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

7.1.4.4.7 Structure Removal Using SPMT

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

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

7.1.5 Project Delivery Methods/Bidding Process

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

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

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

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

WisDOT policy item:

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

7.2 ABC Decision-Making Guidance

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

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

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

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

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

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

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

Figure 7.2-1

ABC Decision-Making Matrix

7.2.1 Descriptions of Terms in ABC Decision-Making Matrix

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

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

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

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

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

Table 7.2-1
ABC Decision-Making Matrix Terms

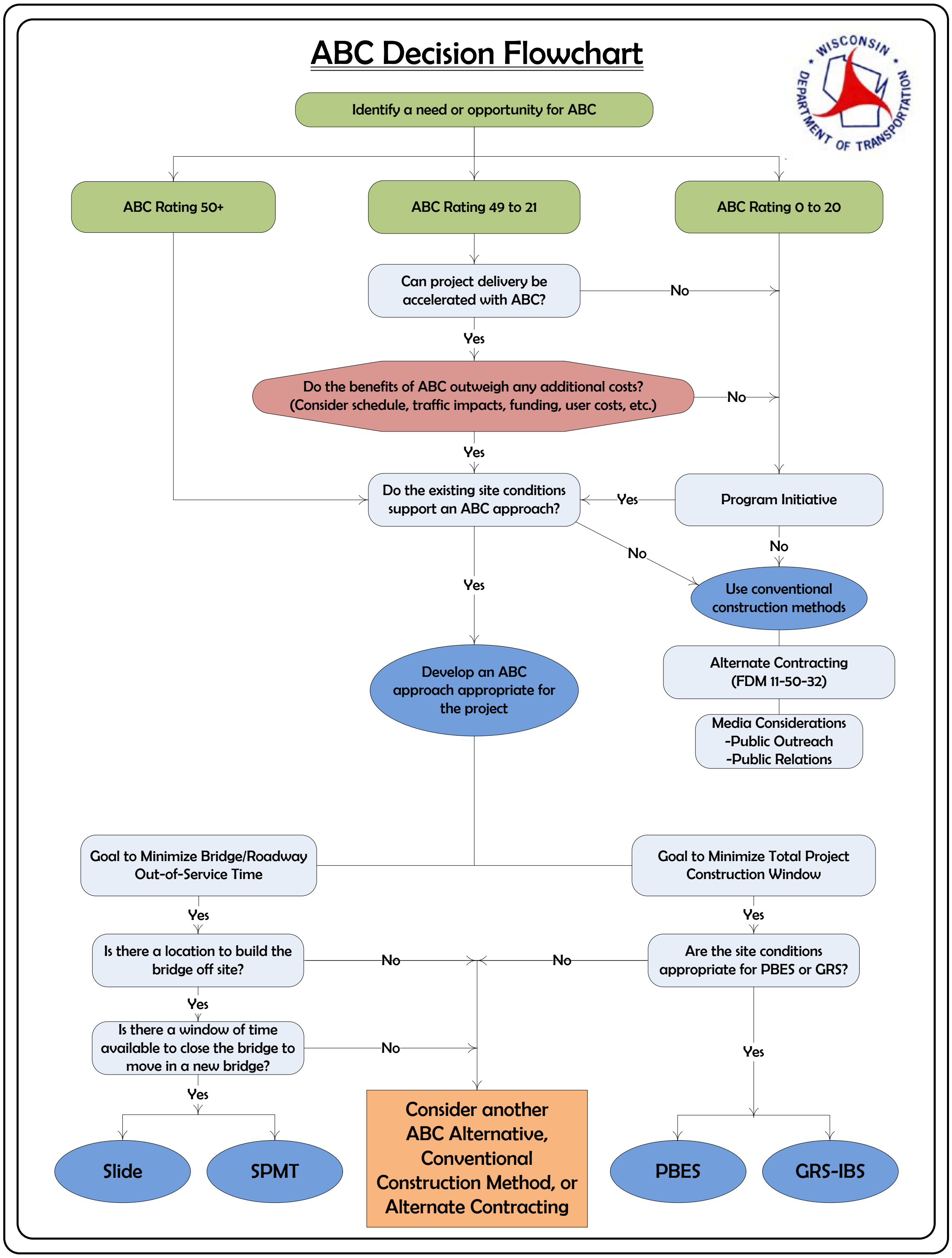

Figure 7.2-2
ABC Decision-Making Flowchart

7.3 References

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

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

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

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Table of Contents

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 www.dot.wisconsin.gov/business/engrserv/approvedprod.htm

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

9.2 Concrete

Concrete is used in many highway structures throughout Wisconsin. Some structure types are composed entirely of concrete, while others have concrete members. Different concrete compressive strengths (f'_c) are used in design and depend on the structure type or the location of the member. Compressive strengths are verified by cylinder tests done on concrete samples taken in the field. The *Standard Specifications* describe the requirements for concrete in Section 501.

Some of the concrete structure types/members and their design strengths for new projects are:

- Decks, Diaphragms, Overlays, Curbs, Parapets, Medians, Sidewalks and Concrete Slab Bridges (f'_c = 4 ksi)
- Other cast-in-place structures such as Culverts, Cantilever Retaining Walls and Substructure units (f'_c = 3.5 ksi)
- Other types of Retaining Walls (f'_c - values as specified in Chapter 14)
- Prestressed "I" girders (f'_c = 6 to 8 ksi)
- Prestressed "Slab and Box" sections (f'_c = 5 ksi)
- Prestressed Deck Panels (f'_c = 6 ksi)

Grade "E" concrete (Low Slump Concrete) is used in overlays for decks and slabs as stated in Section 509.2.

The modulus of elasticity of concrete, E_c, is a function of the unit weight of concrete and its compressive strength **LRFD [C5.4.2.4]**. For a unit weight of 0.150 kcf, the modulus of elasticity is:

f'_c = 3.5 ksi ; E_c = 3600 ksi

f'_c = 4 ksi ; E_c = 3800 ksi

For prestressed concrete members, the value for E_c is based on studies in the field and is calculated as shown in 19.3.3.8.

The modulus of rupture for concrete, f_r, is a function of the concrete strength and is described in **LRFD [5.4.2.6]**. The coefficient of thermal expansion for normal weight concrete is 6×10^{-6} in/in/°F per **LRFD [5.4.2.2]**.

Air entraining admixture is added to concrete to provide durability for exposure to freeze and thaw conditions. Other concrete admixtures used are set retarding and water reducing admixtures. These are covered in Section 501 of the *Standard Specifications.*

9.3 Reinforcement Bars

Reinforced concrete structures and concrete members are designed using Grade 60 deformed bar steel with a minimum yield strength of 60 ksi. The modulus of elasticity, E_s , for steel reinforcing is 29,000 ksi. Reinforcement may be epoxy coated and this is determined by its location in the structure as described below. Adequate concrete cover and epoxy coating of reinforcement contribute to the durability of the reinforced concrete structure. The *Standard Specifications* describe the requirements for steel reinforcement and epoxy coating in Section 505.

Epoxy coated bars shall be used for both top and bottom reinforcement on all new decks, deck replacements, concrete slab superstructures, structural approach slabs and top slab of culverts (with no fill on top). They shall be used in other superstructure elements such as curbs, parapets, medians, sidewalks, diaphragms and pilasters. Some of the bars in prestressed girders are epoxy coated and are specified in the Chapter 19 - Standards. Also use coated bars for sign bridge footings.

Use epoxy coated bar steel on all piers detailed with expansion joints and on all piers at grade separations. Use epoxy coated bars down to the top of the footing elevation.

At all abutments, epoxy coated bars shall be used for parapets on wing walls. For A3/A4 abutments use epoxy coated bars for the paving block and the abutment backwall, and for A1(fixed) coat the dowel bars. For all abutments use epoxy coated bars in the wing walls.

Welding of bar steel is not permitted unless approved by the Bureau of Structures or used in an approved butt splice as stated in Section 505.3.3.3 of the *Standard Specifications*. Test results indicate that the fatigue life of steel reinforcement is reduced by welding to them. Supporting a deck joint by welding attachments to the bar steel is not permitted. The bar steel mat does not provide adequate stiffness to support deck joints or similar details during the deck pour and maintain the proper joint elevations.

The minimum and maximum spacing of reinforcement, and spacing between bar layers is provided in **LRFD [5.10.3.1, 5.10.3.2]**. Use minimum and maximum values shown on Standards where provided.

Bridge plans show the quantity of bar steel required for the structure. Details are not provided for bar chairs or other devices necessary to support the reinforcement during the placement of the concrete. This information is covered by the *Standard Specifications* in Section 505.3.4 and these devices are part of the bid quantity.

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete as stated in **LRFD [5.10.8]**.

When determining the anchorage requirements for bars, consider the bar size, the development length for straight bars and the development length for standard hooks. Note in Table 9.9-1 and Table 9.9-2 that smaller bars require considerably less development length than larger bars and the development length is also less if the bar spacing is 6 inches or more. By detailing smaller bars to get the required area and providing a spacing of 6 inches or more, less steel is used. Bar hooks can reduce the required bar development lengths, however the hooks may cost more to fabricate. In cases such as footings for columns or retaining walls, hooks may be the only practical solution because of the concrete depth available for developing the capacity of the bars.

Fabricators stock all bar sizes in 60 foot lengths. For ease of handling, the detailed length for #3 and #4 bars is limited to 45 feet. Longer bars may be used for these bar sizes at the discretion of the designer, when larger quantities are required for the structure. All other bar sizes are detailed to a length not to exceed 60 feet, except for vertical bars. Bars placed in a vertical position are detailed to match optional construction joint spacing plus lap. The location of optional horizontal construction joints in pier shafts or columns will generally determine the length of vertical bars in piers. All bars are detailed to the nearest inch.

The number of bars in a bundle shall not exceed four, except in flexural members the bars larger than #11 shall not exceed two in any one bundle. Individual bars in a bundle, cut off within the span of a member, shall be terminated at different points with at least a 40-bar diameter stagger. Where spacing limitations are based on bar size, bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area **LRFD [5.10.3.1.5]**.

Stainless steel deformed reinforcement meeting the requirements of ASTM A955 has been used on a limited basis with the approval of the Bureau of Structures. It has been used in bridge decks, parapets and in the structural approach slabs at the ends of the bridge. Fabricators typically stock #6 bars and smaller in 60 foot lengths and #7 bars and larger in 40 foot lengths. Follow the guidance above for selecting bar lengths based on ease of handling.

9.3.1 Development Length and Lap Splices for Deformed Bars

Table 9.9-1 and Table 9.9-2 provide the development length, ℓ_d , for straight bars and the required lap length of spliced tension bars according to **LRFD [5.11.2.1, 5.11.5.3]**. The basic development length, ℓ_{db} , is a function of bar area, A_b , bar diameter, d_b , concrete strength, f'_c and yield strength of reinforcement, f_y. The basic development length is multiplied by applicable modification factors to produce the required development length, ℓ_d. The lap lengths for spliced tension bars are equal to a factor multiplied times the development length, ℓ_d . The factor applied depends on the classification of the splice; Class A, B or C. The class selected is a function of the numbers of bars spliced at a given location and the ratio of the area of reinforcement provided to the area required. The values for development length (required embedment) are equal to Class "A" splice lengths shown in these tables. Table 9.9-1 gives the development lengths and required lap lengths for a concrete compressive strength of f'_c = 3.5 ksi and a reinforcement yield strength of f_y = 60 ksi. Table 9.9-2 gives these same lengths for a concrete compressive strength of f'_c = 4 ksi and a reinforcement yield strength of f_y = 60 ksi. In tensile stress zones the maximum allowable change in bar size at a lap is three bar sizes. The spacing of lap splice reinforcement is provided in **LRFD [5.10.3.1.4]**, but values on Standards should be used where provided.

The development length of individual bars within a bundle, shall be that for the individual bar, increased by 20% for a three-bar bundle and by 33% for a four-bar bundle **LRFD [5.11.2.3]**. For determining the modification factors specified in **LRFD [5.11.2.1.2, 5.11.2.1.3]**, a unit of bundled bars shall be treated as a single bar of a diameter determined from the equivalent total area.

Lap splices within bundles shall be as specified in **LRFD [5.11.2.3]**. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced **LRFD [5.11.5.2.1]**.

Hook and embedment requirements for transverse (shear) reinforcement are stated in **LRFD [5.11.2.6.2]**. The lap length for pairs of U-stirrups or ties that are placed to form a closed unit shall be considered properly anchored and spliced where lengths of laps are not less than 1.7 ℓ_d **LRFD [5.11.2.6.4]**. In members not less than 18 inches deep, the length of the stirrup leg for anchoring closed stirrup splices is described in **LRFD [5.11.2.6.4]**.

The Bureau of Structures interprets the lap length to be used for temperature and distribution reinforcement to be a Class “A” splice (using “top” or “others”, as appropriate). See Table 9.9-1 and Table 9.9-2 for definition of “top” bars.

The required development length, ℓ_{dh}, for bars in tension terminating in a standard hook is detailed in **LRFD [5.11.2.4]**. This length increases with the bar size. The basic development length, ℓ_{hb}, for a hooked bar is a function of bar diameter, d_b, and concrete strength, f'_c. The basic development length is multiplied by applicable modification factors to produce the required development length, ℓ_{dh}.

Embedment depth is increased for dowel bars (with hooked ends) that run from column or retaining wall into the footing, if the hook does not rest on top of the bar steel mat in the bottom of the footing. This is a construction detail which is the preferred method for anchoring these bars before the concrete is placed.

Dowel bars are used as tensile reinforcement to tie columns or retaining walls to their footings. The amount of bar steel can be reduced by varying the dowel bar lengths projecting above the footing, so that only half the bars are spliced in the same plane. This is a consideration for long retaining walls and for some columns. This allows a Class “B” splice to be used, as opposed to a Class “C” splice where equal length dowel bars are used and all bars are spliced in the same plane.

The length of lap, ℓ_c , for splices in compression is provided in **LRFD [5.11.5.5.1]**.

9.3.2 Bends and Hooks for Deformed Bars

Figure 9.9-1 shows standard hook and bend details for development of longitudinal tension reinforcement. Figure 9.9-2 shows standard hook and bend details for transverse reinforcement (open stirrups and ties). Figure 9.9-3 shows details for transverse reinforcement (closed stirrups). Dimensions for the bending details are shown as out to out of bar, as stated in the *Standard Specifications* Section 505.3.2. The diameter of a bend, measured on the inside of the bar for a standard bend is specified in **LRFD [5.10.2.3]**. Where a larger bend radius is required (non-standard bend) show the inside bend radius on the bar detail. When computing total bar lengths account for the accumulation in length in the bends. Use the figures mentioned above to account for accumulation in length for standard hooks and bends. One leg of bent bars is not dimensioned so that the tolerance for an error in the length due to bending is placed there. Fabrication tolerances for bent bars are specified in the *Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practices* or the *American Concrete Institute (ACI) Detailing Manual* as stated in Section 505.2.1 of the *Standard Specifications*.

9.7 Miscellaneous Materials

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

- Galvanized or stainless steel reinforcing bars
- Waterproofing membrane with bituminous concrete overlay
- Microsilica modified concrete or polymer impregnated concrete
- Low slump concrete overlays
- Low-viscosity crack sealer
- Cathodic protection systems with surface overlays

Other materials or products used on highway structures are:

- Downspouts for Type GC and H drains may be fabricated from fiberglass conforming to ASTM D2996, Grade 1, Class A.
- Elastomeric bearing pads (non-laminated) are primarily used with prestressed "I" girders at fixed abutments and piers and at semi-expansion abutments. They are also used with prestressed "slab and box" sections at all supports. The requirements for these pads are described in Section 506.2.6.4 of the *Standard Specifications*.
- Elastomeric bearing pads (laminated) are primarily used with prestressed "I" girders at expansion supports. The requirements for these pads are described in Section 506.2.6.5 of the *Standard Specifications*.
- Preformed fillers are placed vertically in the joint between wing and diaphragm in A1 and A5 abutments, in the joint between wing and barrel in box culverts and at expansion joints in concrete cast-in-place retaining walls. Preformed fillers are placed along the front top surface of A1 and A5 abutments, along the outside top surfaces of fixed piers and under flanges between elastomeric bearing pad (non-laminated) and top edge of support. Cork filler is placed vertically on semi-expansion abutments. The requirements for fillers are described in Section 502.2.7 of the *Standard Specifications*.
- Polyethelene sheets are placed on the top surface of semi-expansion abutments to allow movement of the superstructure across the bearing surface. They are also placed between the structural approach slab and the subgrade, and the approach slab and its footing.

- Rubberized waterproofing membranes are used to seal horizontal and vertical joints at the backface of abutments, culverts and concrete cast-in-place retaining walls. See Section 5.16.2.3 of the *Standard Specifications*.

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

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

- Preformed Fabric, Class A, has been used as a bearing pad under steel bearings. The requirement for this material is given in Section 506.2.6.3 of the *Standard Specifications*.

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

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

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

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

9.8 Painting

All highway grade separation structures require steel girders to be painted because unpainted steel is subject to additional corrosion from vehicle salt spray. Additional discussion on painting is presented in Chapter 24 – Steel Girder Structures. The current paint system used for I-girders is the three-coat epoxy system specified in Section 517 of the *Standard Specifications*. Tub girders utilize a two-coat polysiloxane system, which includes painting of the inside of the tubs.

Recommended standard colors and paint color numbers for steel girders in Wisconsin in accordance with Federal Standard No. 595B as printed are:

White (For Inside of Box Girders)	#27925
Blue (Medium Sky Blue Tone)	#25240
[1] Brown (Similar to Weathering Steel)	#20059
Gray (Light Gray)	#26293
Green (Medium Tone)	#24260
Reddish-Brown (Red Brick Tone)	#20152
Gray (Dark Gray-DNR Request)	#26132
Black	#27038

Table 9.8-1
Standard Colors for Steel Girders

[1] Shop applied color for weathering steel.

Federal Standard No. 595B can be found at www.colorserver.net/

All steel bearing components which are not welded to the girder or do not have a Teflon or bronze surface, and anchor bolts shall be galvanized. In addition to galvanizing, some bearing components may also be field or shop painted as noted in the Standards for Chapter 27 – Bearings.

All new structural steel is blast cleaned including weathering steel. It has been shown that paint systems perform well over a longer period of time with proper surface preparation. The blast cleaned surface is a very finely pitted surface with pit depths of 1 ½ mils.

Corrosion of structural steel occurs if the agents necessary to form a corrosion cell are present. A corrosion cell is similar to a battery in that current flows from the anode to the cathode. As the current flows, corrosion occurs at the anode and materials expand. Water carries the electrical current between the anode and cathode. If there is salt in the water, the current travels much faster and the rate of corrosion is accelerated. Oxygen combines with the materials to help form the anodic corrosion cell.

The primary reason for painting steel structures is for the protection of the steel surface. Appearance is a secondary function that is maintained by using compatible top coatings over epoxy systems. Regarding appearance with respect to color retention, black is good, blues and greens are decent, and reddish browns are acceptable, but not the best. Reds are highly discouraged and should not be used.

Paint applied to the steel acts as a moisture barrier. It prevents the water from contacting the steel and then a corrosion cell cannot be formed. When applying a moisture barrier, it is important to get an adhering, uniform thickness as well as an adequate thickness. The film thickness of paint wears with age until it is finally depleted. At this point the steel begins to corrode as moisture is now present in the corrosion cell. If paint is applied too thick, it may crack and/or check due to temperature changes and allow moisture to contact the steel long before the film thickness wears down.

The paint inspector uses a paint gauge to randomly measure the film thickness of the paint according to specifications. Wet film thickness is measured and it is always thicker than the dry film thickness. A vehicle is added to the paint solids so that the solids can be applied to a surface and then it evaporates leaving only the solids on the surface. The percent of solids in a gallon of paint gives an estimate of the wet to dry film thickness ratio.

Refer to Section 1.3.14 of the *Wisconsin Structure Inspection Manual* for the criteria covering spot painting versus complete painting of existing structures. This Section provides information for evaluating the condition of a paint system and recommended maintenance.

Recommended standard colors and color numbers for concrete in Wisconsin in accordance with Federal Standard No. 595B as printed are:

Pearl Gray	#26622
Medium Tan	#33446
Gray Green	#30372
Dark Brown	#30140
Dawn Mist (Grayish Brown)	#36424
Lt. Coffee (Creamy Brown)	#33722

Table 9.8-2
Standard Colors for Concrete

Most paints require concrete to be a minimum of 30 days old before application. This should be considered when specifying completion times for contracts.

BAR SIZE	BAR WEIGHT (lbs/ft)	NOM. DIA (in)	NOM. AREA (in^2)	NUMBER OF BARS								
				2	3	4	5	6	7	8	9	10
4	0.668	0.500	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60	1.80	2.00
5	1.043	0.625	0.31	0.62	0.93	1.24	1.55	1.86	2.17	2.48	2.79	3.10
6	1.502	0.750	0.44	0.88	1.32	1.76	2.20	2.64	3.08	3.52	3.96	4.40
7	2.044	0.875	0.60	1.20	1.80	2.40	3.00	3.60	4.20	4.80	5.40	6.00
8	2.670	1.000	0.79	1.58	2.37	3.16	3.95	4.74	5.53	6.32	7.11	7.90
9	3.400	1.128	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00
10	4.303	1.270	1.27	2.54	3.81	5.08	6.35	7.62	8.89	10.16	11.43	12.70
11	5.313	1.410	1.56	3.12	4.68	6.24	7.80	9.36	10.92	12.48	14.04	15.60

Table 9.9-3
Bar Areas Per Number of Bars (in^2)

BAR SIZE	4 ½“	5”	5 ½“	6”	6 ½“	7”	7 ½”	8”	8 ½”	9”	10”	11”	12”
4	0.52	0.47	0.43	0.39	0.36	0.34	0.31	0.29	0.28	0.26	0.24	0.21	0.20
5	0.82	0.74	0.67	0.61	0.57	0.53	0.49	0.46	0.43	0.41	0.37	0.33	0.31
6	1.18	1.06	0.96	0.88	0.82	0.76	0.71	0.66	0.62	0.59	0.53	0.48	0.44
7	1.60	1.44	1.31	1.20	1.11	1.03	0.96	0.90	0.85	0.80	0.72	0.66	0.60
8	2.09	1.88	1.71	1.57	1.45	1.35	1.26	1.18	1.11	1.05	0.94	0.86	0.79
9	---	2.40	2.18	2.00	1.85	1.71	1.60	1.50	1.41	1.33	1.20	1.09	1.00
10	---	3.04	2.76	2.53	2.34	2.17	2.02	1.90	1.79	1.69	1.52	1.38	1.27
11	---	3.75	3.41	3.12	2.88	2.68	2.50	2.34	2.21	2.08	1.87	1.70	1.56

Table 9.9-4
Area of Bar Reinf. (in^2 / ft) vs. Spacing of Bars (in)

9.10 References

1. Ghorbanpoor, A., Kriha, B., Reshadi, R. *Aesthetic Coating for Steel Bridge Components – Amended Study*. S.1.: Wisconsin Department of Transportation, Final Report No. 0092-11-07, 2015.

9.11 Appendix - Draft Bar Tables

The following Draft Bar Tables are provided for information only. We expect the tables to be moved into the main text of Chapter 9 in July of 2016, and at that time to begin their use. We are delaying their use to allow time for modification of details and software that are affected.

The 2015 Interim Revisions to the AASHTO LRFD Bridge Design Specifications (7th Edition), modified the tension development lengths and tension lap lengths for straight deformed bars as follows:

The tension development length, ℓ_d ,shall not be less than the product of the basic tension development length, ℓ_{db} , and the appropriate modification factors, λ_i . **LRFD [5.11.2.1.1]**

$$\ell_d = \ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{lw} \cdot \lambda_{rc} \cdot \lambda_{er})$$

in which: $\ell_{db} = 2.4 \cdot d_b \cdot [f_y / (f'_c)^{1/2}]$

where:

ℓ_{db} = basic development length (in.)

λ_{rl} = reinforcement location factor

λ_{cf} = coating factor

λ_{lw} = lightweight concrete factor

λ_{rc} = reinforcement confinement factor

λ_{er} = excess reinforcement factor

f_y = specified yield strength of reinforcing bars (ksi)

d_b = diameter of bar (in.)

f'_c = specified compressive strength of concrete (ksi)

Top bars will continue to refer to horizontal bars placed with more than 12" of fresh concrete cast below it. Bars not meeting this criteria will be referred to as Others.

Per **LRFD [5.11.5.3.1]**, there are two lap splice classes, Class A and Class B.

- Class A lap splice ……………………1.0 ℓ_d
- Class B lap splice ………………….. 1.3 ℓ_d

The criteria for where to apply each Class is covered in the above reference.

Draft Table

Epoxy Coated (f'$_c$ = 4,000 psi; f$_y$ = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-6"	1'-6"	1'-11"	1'-11"
	2.0"	1'-6"	1'-6"	1'-11"	1'-11"
	≥ 2.5"	1'-6"	1'-6"	1'-11"	1'-11"
5	1.5"	2'-3"	2'-3"	3'-0"	3'-0"
	2.0"	2'-3"	1'-10"	3'-0"	2'-4"
	≥ 2.5"	2'-3"	1'-10"	3'-0"	2'-4"
6	1.5"	2'-11"	2'-9"	3'-10"	3'-7"
	2.0"	2'-11"	2'-9"	3'-10"	3'-7"
	≥ 2.5"	2'-11"	2'-2"	3'-10"	2'-10"
7	1.5"	3'-7"	3'-7"	4'-8"	4'-8"
	2.0"	3'-6"	3'-2"	4'-6"	4'-2"
	≥ 2.5"	3'-6"	3'-2"	4'-6"	4'-2"
8	1.5"	4'-6"	4'-6"	5'-11"	5'-11"
	2.0"	4'-6"	3'-8"	5'-11"	4'-9"
	≥ 2.5"	4'-6"	3'-8"	5'-11"	4'-9"
9	1.5"	5'-9"	5'-7"	7'-5"	7'-4"
	2.0"	5'-9"	4'-6"	7'-5"	5'-10"
	≥ 2.5"	5'-9"	4'-1"	7'-5"	5'-4"
10	1.5"	7'-4"	6'-9"	9'-7"	8'-10"
	2.0"	7'-4"	5'-6"	9'-7"	7'-2"
	≥ 2.5"	7'-4"	4'-10"	9'-7"	6'-3"
11	1.5"	9'-1"	8'-2"	11'-9"	10'-7"
	2.0"	9'-1"	6'-8"	11'-9"	8'-7"
	≥ 2.5"	9'-1"	6'-0"	11'-9"	7'-9"

Epoxy Coated (f'$_c$ = 4,000 psi; f$_y$ = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-11"	1'-11"	2'-6"	2'-6"
	2.0"	1'-11"	1'-11"	2'-6"	2'-6"
	≥ 2.5"	1'-11"	1'-11"	2'-6"	2'-6"
5	1.5"	2'-7"	2'-7"	3'-4"	3'-4"
	2.0"	2'-7"	2'-4"	3'-4"	3'-1"
	≥ 2.5"	2'-7"	2'-4"	3'-4"	3'-1"
6	1.5"	3'-4"	3'-1"	4'-4"	4'-0"
	2.0"	3'-4"	3'-1"	4'-4"	4'-0"
	≥ 2.5"	3'-4"	2'-10"	4'-4"	3'-8"
7	1.5"	4'-1"	4'-1"	5'-3"	5'-3"
	2.0"	4'-0"	3'-7"	5'-2"	4'-8"
	≥ 2.5"	4'-0"	3'-7"	5'-2"	4'-8
8	1.5"	5'-2"	5'-2"	6'-8"	6'-8"
	2.0"	5'-2"	4'-1"	6'-8"	5'-4"
	≥ 2.5"	5'-2"	4'-1"	6'-8"	5'-4"
9	1.5"	6'-6"	6'-4"	8'-5"	8'-3"
	2.0"	6'-6"	5'-1"	8'-5"	6'-7"
	≥ 2.5"	6'-6"	4'-8"	8'-5"	6'-0"
10	1.5"	8'-4"	7'-8"	10'-10"	10'-0"
	2.0"	8'-4"	6'-3"	10'-10"	8'-1"
	≥ 2.5"	8'-4"	5'-6"	10'-10"	7'-1"
11	1.5"	10'-3"	9'-3"	13'-4"	12'-0"
	2.0"	10'-3"	7'-6"	13'-4"	9'-9"
	≥ 2.5"	10'-3"	6'-10"	13'-4"	8'-10"

Draft Table

Uncoated (f'_c = 4,000 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-3"	1'-3"	1'-7"	1'-7"
	2.0"	1'-3"	1'-3"	1'-7"	1'-7"
	≥ 2.5"	1'-3"	1'-3"	1'-7"	1'-7"
5	1.5"	1'-6"	1'-6"	2'-0"	2'-0"
	2.0"	1'-6"	1'-6"	2'-0"	2'-0"
	≥ 2.5"	1'-6"	1'-6"	2'-0"	2'-0"
6	1.5"	2'-0"	1'-10"	2'-7"	2'-4"
	2.0"	2'-0"	1'-10"	2'-7"	2'-4"
	≥ 2.5"	2'-0"	1'-10"	2'-7"	2'-4"
7	1.5"	2'-5"	2'-5"	3'-1"	3'-1"
	2.0"	2'-4"	2'-2"	3'-0"	2'-9"
	≥ 2.5"	2'-4"	2'-2"	3'-0"	2'-9"
8	1.5"	3'-0"	3'-0	3'-11"	3'-11"
	2.0"	3'-0"	2'-5"	3'-11"	3'-2"
	≥ 2.5"	3'-0"	2'-5"	3'-11"	3'-2"
9	1.5"	3'-10"	3'-9"	5'-0"	4'-10"
	2.0"	3'-10"	3'-0"	5'-0"	3'-11"
	≥ 2.5"	3'-10"	2'-9"	5'-0"	3'-7"
10	1.5"	4'-11"	4'-6"	6'-4"	5'-11"
	2.0"	4'-11"	3'-8"	6'-4"	4'-9"
	≥ 2.5"	4'-11"	3'-3"	6'-4"	4'-2"
11	1.5"	6'-0"	5'-5"	7'-10"	7'-1"
	2.0"	6'-0"	4'-5"	7'-10"	5'-9"
	≥ 2.5"	6'-0"	4'-0"	7'-10"	5'-2"

Uncoated (f'_c = 4,000 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-7"	1'-7"	2'-1"	2'-1"
	2.0"	1'-7"	1'-7"	2'-1"	2'-1"
	≥ 2.5"	1'-7"	1'-7"	2'-1"	2'-1"
5	1.5"	2'-0"	2'-0"	2'-7"	2'-7"
	2.0"	2'-0"	2'-0"	2'-7"	2'-7"
	≥ 2.5"	2'-0"	2'-0"	2'-7"	2'-7"
6	1.5"	2'-7"	2'-4"	3'-4"	3'-1"
	2.0"	2'-7"	2'-4"	3'-4"	3'-1"
	≥ 2.5"	2'-7"	2'-4"	3'-4"	3'-1"
7	1.5"	3'-1"	3'-1"	4'-0"	4'-0"
	2.0"	3'-0"	2'-9"	3'-11"	3'-7"
	≥ 2.5"	3'-0"	2'-9"	3'-11"	3'-7"
8	1.5"	3'-11"	3'-11"	5'-1"	5'-1"
	2.0"	3'-11"	3'-2"	5'-1"	4'-1"
	≥ 2.5"	3'-11"	3'-2"	5'-1"	4'-1"
9	1.5"	5'-0"	4'-10"	6'-5"	6'-4"
	2.0"	5'-0"	3'-11"	6'-5"	5'-1"
	≥ 2.5"	5'-0"	3'-7"	6'-5"	4'-7"
10	1.5"	6'-4"	5'-11"	8'-3"	7'-8"
	2.0"	6'-4"	4'-9"	8'-3"	6'-3"
	≥ 2.5"	6'-4"	4'-2"	8'-3"	5'-5"
11	1.5"	7'-10"	7'-1"	10'-2"	9'-2"
	2.0"	7'-10"	5'-9"	10'-2"	7'-6"
	≥ 2.5"	7'-10"	5'-2"	10'-2"	6'-9"

Draft Table

Epoxy Coated (f'_c = 3,500 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-7"	1'-7"	2'-0"	2'-0"
	2.0"	1'-7"	1'-7"	2'-0"	2'-0"
	≥ 2.5"	1'-7"	1'-7"	2'-0"	2'-0"
5	1.5"	2'-5"	2'-5"	3'-2"	3'-2"
	2.0"	2'-5"	1'-11"	3'-2"	2'-6"
	≥ 2.5"	2'-5"	1'-11"	3'-2"	2'-6"
6	1.5"	3'-2"	2'-11"	4'-1"	3'-9"
	2.0"	3'-2"	2'-11"	4'-1"	3'-9"
	≥ 2.5"	3'-2"	2'-4"	4'-1"	3'-0"
7	1.5"	3'-10"	3'-10"	5'-0"	5'-0"
	2.0"	3'-9"	3'-5"	4'-10"	4'-5"
	≥ 2.5"	3'-9"	3'-5"	4'-10"	4'-5"
8	1.5"	4'-10"	4'-10"	6'-3"	6'-3"
	2.0"	4'-10"	3'-11"	6'-3"	5'-0"
	≥ 2.5"	4'-10"	3'-11"	6'-3"	5'-0"
9	1.5"	6'-1"	6'-0"	7'-11"	7'-10"
	2.0"	6'-1"	4'-10"	7'-11"	6'-3"
	≥ 2.5"	6'-1"	4'-4"	7'-11"	5'-8"
10	1.5"	7'-10"	7'-3"	10'-2"	9'-5"
	2.0"	7'-10"	5'-11"	10'-2"	7'-8"
	≥ 2.5"	7'-10"	5'-2"	10'-2"	6'-8"
11	1.5"	9'-8"	8'-9"	12'-7"	11'-4"
	2.0"	9'-8"	7'-1"	12'-7"	9'-2"
	≥ 2.5"	9'-8"	6'-5"	12'-7"	8'-4"

Epoxy Coated (f'_c = 3,500 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	2'-0"	2'-0"	2'-8"	2'-8"
	2.0"	2'-0"	2'-0"	2'-8"	2'-8"
	≥ 2.5"	2'-0"	2'-0"	2'-8"	2'-8"
5	1.5"	2'-9"	2'-9"	3'-7"	3'-7"
	2.0"	2'-9"	2'-6"	3'-7"	3'-3"
	≥ 2.5"	2'-9"	2'-6"	3'-7"	3'-3"
6	1.5"	3'-7"	3'-4"	4'-7"	4'-3"
	2.0"	3'-7"	3'-4"	4'-7"	4'-3"
	≥ 2.5"	3'-7"	3'-0"	4'-7"	3'-11"
7	1.5"	4'-4"	4'-4"	5'-7"	5'-7"
	2.0"	4'-3"	3'-10"	5'-6"	5'-0"
	≥ 2.5"	4'-3"	3'-10"	5'-6"	5'-0"
8	1.5"	5'-6"	5'-6"	7'-1"	7'-1"
	2.0"	5'-6"	4'-5"	7'-1"	5'-8"
	≥ 2.5"	5'-6"	4'-5"	7'-1"	5'-8"
9	1.5"	6'-11"	6'-10"	9'-0"	8'-10"
	2.0"	6'-11"	5'-5"	9'-0"	7'-1"
	≥ 2.5"	6'-11"	4'-11"	9'-0"	6'-5"
10	1.5"	8'-11"	8'-2"	11'-7"	10'-8"
	2.0"	8'-11"	6'-8"	11'-7"	8'-8"
	≥ 2.5"	8'-11"	5'-10"	11'-7"	7'-7"
11	1.5"	10'-11"	9'-10"	14'-3"	12'-10"
	2.0"	10'-11"	8'-0"	14'-3"	10'-5"
	≥ 2.5"	10'-11"	7'-3"	14'-3"	9'-5"

Draft Table

Uncoated (f'_c = 3,500 psi; f_y = 60,000 psi)

Basic Lap - Others					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-4"	1'-4"	1'-8"	1'-8"
	2.0"	1'-4"	1'-4"	1'-8"	1'-8"
	≥ 2.5"	1'-4"	1'-4"	1'-8"	1'-8"
5	1.5"	1'-8"	1'-8"	2'-1"	2'-1"
	2.0"	1'-8"	1'-8"	2'-1"	2'-1"
	≥ 2.5"	1'-8"	1'-8"	2'-1"	2'-1"
6	1.5"	2'-1"	1'-11"	2'-9"	2'-6"
	2.0"	2'-1"	1'-11"	2'-9"	2'-6"
	≥ 2.5"	2'-1"	1'-11"	2'-9"	2'-6"
7	1.5"	2'-7"	2'-7"	3'-4"	3'-4"
	2.0"	2'-6"	2'-3"	3'-3"	2'-11"
	≥ 2.5"	2'-6"	2'-3"	3'-3"	2'-11"
8	1.5"	3'-3"	3'-3	4'-2"	4'-2"
	2.0"	3'-3"	2'-7"	4'-2"	3'-4"
	≥ 2.5"	3'-3"	2'-7"	4'-2"	3'-4"
9	1.5"	4'-1"	4'-0"	5'-4"	5'-2"
	2.0"	4'-1"	3'-3"	5'-4"	4'-2"
	≥ 2.5"	4'-1"	2'-11"	5'-4"	3'-10"
10	1.5"	5'-3"	4'-10"	6'-10"	6'-3"
	2.0"	5'-3"	3'-11"	6'-10"	5'-1"
	≥ 2.5"	5'-3"	3'-5"	6'-10"	4'-6"
11	1.5"	6'-5"	5'-10"	8'-5"	7'-7"
	2.0"	6'-5"	4'-9"	8'-5"	6'-2"
	≥ 2.5"	6'-5"	4'-3"	8'-5"	5'-7"

Uncoated (f'_c = 3,500 psi; f_y = 60,000 psi)

Horizontal Lap w/ >12" Concrete Cast Below - Top					
Bar Size	Min. Cover	Class A (1.0 ℓ_d)		Class B (1.3 ℓ_d)	
		s < 6" cts.	s ≥ 6" cts.	s < 6" cts.	s ≥ 6" cts.
4	1.5"	1'-8"	1'-8"	2'-2"	2'-2"
	2.0"	1'-8"	1'-8"	2'-2"	2'-2"
	≥ 2.5"	1'-8"	1'-8"	2'-2"	2'-2"
5	1.5"	2'-1"	2'-1"	2'-9"	2'-9"
	2.0"	2'-1"	2'-1"	2'-9"	2'-9"
	≥ 2.5"	2'-1"	2'-1"	2'-9"	2'-9"
6	1.5"	2'-9"	2'-6"	3'-6"	3'-3"
	2.0"	2'-9"	2'-6"	3'-6"	3'-3"
	≥ 2.5"	2'-9"	2'-6"	3'-6"	3'-3"
7	1.5"	3'-4"	3'-4"	4'-4"	4'-4"
	2.0"	3'-3"	2'-11"	4'-2"	3'-10"
	≥ 2.5"	3'-3"	2'-11"	4'-2"	3'-10"
8	1.5"	4'-2"	4'-2"	5'-5"	5'-5"
	2.0"	4'-2"	3'-4"	5'-5"	4'-4"
	≥ 2.5"	4'-2"	3'-4"	5'-5"	4'-4"
9	1.5"	5'-4"	5'-2"	6'-11"	6'-9"
	2.0"	5'-4"	4'-2"	6'-11"	5'-5"
	≥ 2.5"	5'-4"	3'-10"	6'-11"	4'-11"
10	1.5"	6'-10"	6'-3"	8'-10"	8'-2"
	2.0"	6'-10"	5'-1"	8'-10"	6'-8"
	≥ 2.5"	6'-10"	4'-6"	8'-10"	5'-10"
11	1.5"	8'-5"	7'-7"	10'-11"	9'-10"
	2.0"	8'-5"	6'-2"	10'-11"	8'-0"
	≥ 2.5"	8'-5"	5'-7"	10'-11"	7'-3"

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Table of Contents

10.1 General

The purpose of the Geotechnical Investigation is to provide subsurface information for the plans and to develop recommendations for the construction of the structure at reasonable costs versus short and long term performance. The level of Geotechnical Investigation is a function of the type of the structure and the associated performance. For example, a box culvert under a low ADT roadway compared to a multi-span bridge on a major interstate would require a different level of Geotechnical Investigation. The challenge for the geotechnical engineer is to gather subsurface information that will allow for a reasonable assessment of the soil and rock properties compared to the cost of the investigation.

The geotechnical engineer and the structure engineer need to work collectively when evaluating the loads on the structures and the resistance of the soil and rock. The development of the geotechnical investigation and evaluation of the subsurface information requires a degree of engineering judgment. A guide for performing the Geotechnical Investigation is provided in WisDOT Geotechnical Bulletin No. 1, **LRFD [10.4]** and Geotechnical Engineering Circular #5 – Evaluation of Soil and Rock Properties (Sabatini, 2002).

The following structures will require a Geotechnical Investigation:

- Bridges
- Box Culverts
- Retaining Walls
- Non-Standard Sign Structures Foundations
- High Mast Lighting Foundations
- Noise Wall Foundations

5. Mixture of soils - This is the most common case. The soil type with predominant behavior has the controlling name. For example, a soil composed of sand and clay is called sandy clay if the clayey fraction controls behavior.

10.4 Site Investigation Report

The following is a sample of a Site Investigation Report for a two-span bridge and retaining wall. The subsurface exploration drawing is also submitted with the reports.

CORRESPONDENCE/MEMORANDUM ___________________State of Wisconsin

DATE: February 17, 2015

TO: Casey Wierzchowski, P.E.
Southeast Region Soils Engineer

FROM: Jeffrey D Horsfall, P.E.
Geotechnical Engineer

SUBJECT: **Site Investigation Report**
Project I.D. 1060-33-16
B-40-0880
Center Street over USH 45
Milwaukee County

Attached is the Site Investigation Report for the above project.

Please call if you have any questions.

Attachments

cc: Southeast Region (via e-mail)
Bureau of Structures, Structures Design (via e-submit)
Geotechnical File (original)

Site Investigation Report
Project I.D. 1060-33-16
Structure B-40-0880
Center Street over USH 45
Milwaukee County
February 17, 2015

1. GENERAL

The project is Center Street over USH 45, Milwaukee County. The proposed structure has two spans and will replace the existing structure with four spans (B-40-284). The existing structure is supported on spread footings with an allowable bearing capacity of 5,000 psf. The end slope in front of the abutments is to be supported with MSE walls with precast concrete panels. The current topography near the proposed structure is a rolling terrain in an urban area.

The Southeast Region requested that the Geotechnical Engineering Unit evaluate the foundation support for the proposed new structure. The following report presents results of the subsurface investigation, design evaluation, findings, conclusions, and recommendations.

2. SUBSURFACE CONDITIONS

Wisconsin Department of Transportation contracted with Gestra to completed one boring and PSI, Inc. to complete three borings near the proposed structure. Samples were collected in the borings with a method conforming to AASHTO T-206, Standard Penetration Test, in October and November 2014, using automatic hammers (with an efficiency ranging from 84 percent (Gestra) to 69 percent (PSI)). Attachment 1 presents tables showing the summary of subsurface conditions logged in the borings at this site and at the time of drilling for the structure. Attachment 2 presents a figure that illustrates the boring locations and graphical representations of the boring logs. The original borings logs are available at the Geotechnical Engineering Unit and will be made available upon request.

The following describes subsurface conditions in the four borings:

0.7 feet of topsoil or 1.0 feet to 2.0 feet of pavement structure, overlying
0.0 feet to 7.0 feet of brown, dense to very dense, fine to course, sand and gravel, overlying
20.0 feet to 43.0 feet of brown to gray, medium hard, clay, some silt, trace sand, overlying
0.0 feet to 8.0 feet of gray, loose to dense, fine sand, little silt, overlying
0.0 feet to 26.0 feet of gray, medium hard, clay, some silt, trace sand, overlying
Gray, very hard, clay and silt, some gravel

The observed groundwater elevation at the time of drilling ranged from 714 feet to 732 feet as determined by the drillers describing the samples as wet. However, not all of the borings encountered samples that were wet.

3. ANALYSIS ASSUMPTIONS

Foundation analyses are separated into shallow foundations (spread footings) and deep foundations (piling supports). The analyses used the following assumptions:

Shallow Foundation
1. The groundwater elevation ranged from 714 feet to 732 feet.
2. The base of the foundations are at the following elevations

Table 1: Foundation Elevations	
West Abutment	755.9 feet
Pier	733.3 feet
East Abutment	754.4 feet

3. The abutment end slopes are MSE Walls with precast panel facing.
4. The width of the pier footing is 10 feet and the width of the abutment footing is 6 feet.
5. The resistance factor of 0.55 for the factored bearing resistance.

Pile Supported Deep Foundation
1. Soil pressures for displacement piles are based upon a 10 3/4-inch diameter cast-in-place pile.
2. The groundwater elevation ranged from 714 feet to 732 feet.
3. Table 1 presents elevations at the base of the foundations.
4. Nominal soil pressures determined using the computer program APILE.
5. The drivability evaluation was performed using the computer program GRLWEAP.

The design shear strength, cohesion and unit weight for this analyses are presented latter in this report. The values are based upon empirical formulas for internal friction angles using blow counts from the AASHTO T-206 Standard Penetration Test results and the effective overburden pressure for the granular soils, the pocket penetrometer values for the cohesive soils and published values for the bedrock.

4. RESULTS OF ANALYSIS

Shallow Foundation
The results of the shallow foundation evaluation indicated that the factored bearing resistance was 6,000 psf for the west abutment and east abutment and 5,000 psf for the pier. The soils are relatively uniform. The estimated settlement from the bridge loads at the abutments and piers was excessive. The time for settlement would occur over a relatively long period of time.

Deep Foundation
Table 2 shows estimated nominal skin friction and end bearing values for deep foundation pilings.

Drivability
The drivability evaluation used a Delmag D 16-32 diesel hammer to determine if the pile would be overstressed during pile installation. The results of the evaluation indicated that 10 x 42 H-pile at the abutments and the 12 x 53 H-piles at the pier should not be overstressed.

Lateral Earth Pressure
The lateral earth pressure for the backfill material will exert 40 psf for sandy soils. The backfill material will be granular, free draining and locally available.

Table 2: Soil Parameters and Foundation Capacities					
Soil Description	**Friction Angle (degrees)**	**Cohesion (psf)**	**Unit Weight (pcf)**	**Skin Friction[1] (psf)**	**End Bearing[1] (psf)**
B-40-0880 West Abutment (B-1)					
MSE Wall (Elevation 755.9 ft – 738.6 ft)	30	0	120	NA	NA
Clay, gray, trace gravel (Elevation 738.6 ft – 733.4 ft)	0	3,000	125	640	19,100
Clay, gray, trace gravel (Elevation 733.4 ft – 729.4 ft)	0	2,500	120	1,075	21,700
Clay, gray, trace gravel (Elevation 729.4 ft – 717.4 ft)	0	2,000	120	1,370	17,900
Clay and Silt, gray, trace sand and gravel (Elevation 717.4 ft – 705.4 ft)	0	4,500	135	1,210	40,500
Silt, gray, trace sand (Elevation 705.4 ft – 700.4 ft)	0	2,000	120	1,720	17,900
Silt, gray, some sand, trace gravel (Elevation 700.4 ft and below)	0	25,000	135	NA	Refusal
B-40-0880 Pier (B-1Gestra)					
Clay, brown to gray, trace sand, trace gravel (Elevation 733.3 ft – 731.7 ft)	0	2,000	120	340	15,800
Clay, gray, trace gravel (Elevation 731.7 ft – 715.7 ft)	0	3,000	125	930	27,000
Silt, gray, trace gravel (Elevation 715.7 ft – 698.7 ft)	0	3,500	130	495	31,600
Silt, gray, trace gravel (Elevation 698.7 ft – 694.2 ft)	40	0	135	470	417,800
Silt, Sand, Gravel, gray (Elevation 694.2 ft and below)	0	25,000	135	NA	Refusal
1. Skin friction and end bearings vales are the nominal capacities 2. NA - not applicable					

Table 2: Soil Parameters and Foundation Capacities

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Skin Friction[1] (psf)	End Bearing[1] (psf)
B-40-0880 East Abutment (B-2 and B-3)					
MSE Wall (Elevation 754.4 ft – 741.5 ft)	30	0	120	NA	NA
Clay, gray, trace gravel (Elevation 741.5 ft – 732.5 ft)	0	2,500	125	920	22,500
Sand, gray, some silt (Elevation 732.5 ft – 730.5 ft)	36	0	130	620	45,900
Sand, gray, some silt (Elevation 730.5 ft – 728.5 ft)	30	0	115	340	19,700
Clay, gray, trace sand, trace gravel (Elevation 728.5 ft – 717.5 ft)	0	2,500	125	2,380	22,500
Clay, gray, trace sand, trace gravel (Elevation 717.5 ft – 711.0 ft)	0	2,000	120	1,830	17,900
Silt, gray, trace sand (Elevation 711.0 ft – 702.5 ft)	33	0	125	890	50,000
Clay, gray (Elevation 702.5 ft – 692.5 ft)	0	3,000	125	1,730	27,000
Clay and Gravel, gray, some silt (Elevation 692.5 ft and below)	0	25,000	135	NA	Refusal

1. Skin friction and end bearings vales are the nominal capacities
2. NA - not applicable

5. FINDING AND CONCLUSIONS

The following findings and conclusions are based upon the subsurface conditions and analysis:

1. The following describes the subsurface conditions in the four borings:

   0.7 feet of topsoil or 1.0 feet to 2.0 feet of pavement structure, overlying
   0.0 feet to 7.0 feet of brown, dense to very dense, fine to course, sand and gravel, overlying
   20.0 feet to 43.0 feet of brown to gray, medium hard, clay, some silt, trace sand, overlying
   0.0 feet to 8.0 feet of gray, loose to dense, fine sand, little silt, overlying
   0.0 feet to 26.0 feet of gray, medium hard, clay, some silt, trace sand, overlying
   Gray, very hard, clay and silt, some gravel

2. The observed groundwater elevation at the time of drilling ranged from 714 feet to 732 feet as determined by the drillers describing the samples as wet.

3. The results of the shallow foundation evaluation indicated that the factored bearing resistance was 6,000 psf for the west abutment and east abutment and 5,000 psf for the pier. The soils are relatively uniform. The calculations used a resistance factor of 0.55.

4. The estimated settlement from the bridge loads on the shallow foundations would be excessive. The time for settlement would occur over a long period of time.

5. If used the support of the piles will occur in the very hard clay and silt. The pile tip elevation will range from 692 feet to 700 feet. The driven pile lengths will depend upon the type of pile hammer used and actual subsurface conditions encountered.

6. RECOMMENDATIONS

The following recommendations are based upon the findings and conclusions:

1. The recommended support system for the abutments are 10 x 42 H-piles driven to a "Required Driving Resistance" of 180 tons and for the pier footings are 12 x 53 H-piles driven to a "Required Driving Resistance" of 220 tons. Table 3 presents the estimated pile tip elevation for the piles. The actual driven length may be shorter due to the very hard clay.

   Table 3: Estimated H-Pile Tip Elevations

Substructure	Pile Type	Pile Tip Elevation
West Abutment	10 x 42 H-pile	700 feet
Pier	12 x 53 H-pile	694 feet
East Abutment	10 x 42 H-pile	692 feet

2. The field pile capacity should be determined by using the modified Gates dynamic formula. This method will use of a resistance factor of 0.50.

3. Pile points should be used to reduce the potential for damage during driving through the very hard clay and silts.

4. Shallow foundation should not be used based upon the anticipated settlement at the pier and the MSE walls at the abutments.

5. Granular 1 backfill should be used behind the abutments.

Attachment 1
Tables of Subsurface Conditions

Site Investigation Report
Structure B-40-0880
Attachment 1

B-40-0880 Subsurface Conditions							
B-1 Station 19+00.0 22.4 feet left of CE RL				**B-1Gestra Station 20+11.3 38.2 feet left of CE RL**			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count[1]**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
762.6	Pavement Structure			742.7	Pavement Structure	6	14
761.6	Clay, dark brown, trace sand and gravel (fill)	4	7	740.7	Clay, brown to gray, trace sand, trace gravel Qp=1.0 – 3.0	6,9, 9,13	12,17, 16,21
754.1	Clay, brown, some silt, trace sand and gravel Qp=3.0	18	25	731.7	Clay, gray, trace gravel Qp=3.0 – 4.0	9,10, 11,13, 14,12	14,15, 16,18, 19,15
749.6	Clay, gray, trace gravel Qp=1.75 – 3.5	15,13, 14	18,14, 15	715.7	Silt, gray, trace sand Qp=4.0	24,33, 31	27,36, 31
739.6	Clay, gray, trace gravel Qp=3.0 – 3.75	20,14, 18	21,14, 17	698.7	Silt, gray, with gravel Qp=4.5	50/6”	51/6”
733.6	Clay, gray, trace gravel Qp=2.0 – 2.5	23,29	22,26	694.2	Silt, Sand, Gravel, gray Qp=4.5	79, 50/2”	78, 48/2”
729.6	Clay, gray, trace gravel Qp=1.5 – 3.0	13,15, 24,17	12,13, 20,13	689.7	EOB		
717.6	Clay and Silt, gray, trace sand and gravel Qp=3.0 - 4.5+	66,67	49,47				
705.6	Silt, gray, trace sand Qp=1.5	28	18				
700.6	Silt, gray, some sand, trace gravel Qp=4.5+	78,42, 59, 60/4”	49,25, 34, 33/4”				
682.6	EOB						

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring.

B-40-0880 Subsurface Conditions							
B-3 Station 21+10.0 40.6 feet right of CE RL				**B-2 Station 21+14.8 23.3 feet left of CE RL**			
Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
759.4	Topsoil			760.5	Pavement Structure		
758.7	Sand, light brown to brown, fine to course, trace silt and gravel	14,13	32,24	759.5	Sand and Gravel, brown	31	49
755.4	Clay, brown, some silt, trace sand and gravel Qp=4.5 – 4.5+	14,32, 16,50	23,48, 22,65	752.5	Clay and Silt, brown, trace gravel Qp=2.5 – 3.0	11,15	15,18
747.4	Clay, gray, trace sand and gravel Qp=2.5 – 3.25	32,13, 14,15	40,15, 15,15	742.5	Clay, gray, trace gravel Qp=1.75 – 4.5+	18,22, 24,15, 19	19,23, 24,15, 18
730.4	Sand, gray, fine, little silt	29	27	732.5	Sand, gray, some silt	38	35
726.4	Sand, gray, fine, little silt	9	8	730.5	Sand, gray, some silt	9	8
722.4	Silt, gray, little sand, trace clay Qp=3.0	15	13	728.5	Clay, gray, trace sand and gravel Qp=2.5 – 3.0	22,14, 17,20, 21	20,12, 15,17, 17
719.4	EOB			711.0	Silt, gray, trace sand Qp=1.0	38	30
				702.5	Clay, gray Qp=1.75 – 3.0	21,27	16,20
				692.5	Clay and Gravel, gray, some silt Qp=4.5+	117, 108, 60/2'	85, 76, 41/2"
				680.5	EOB		

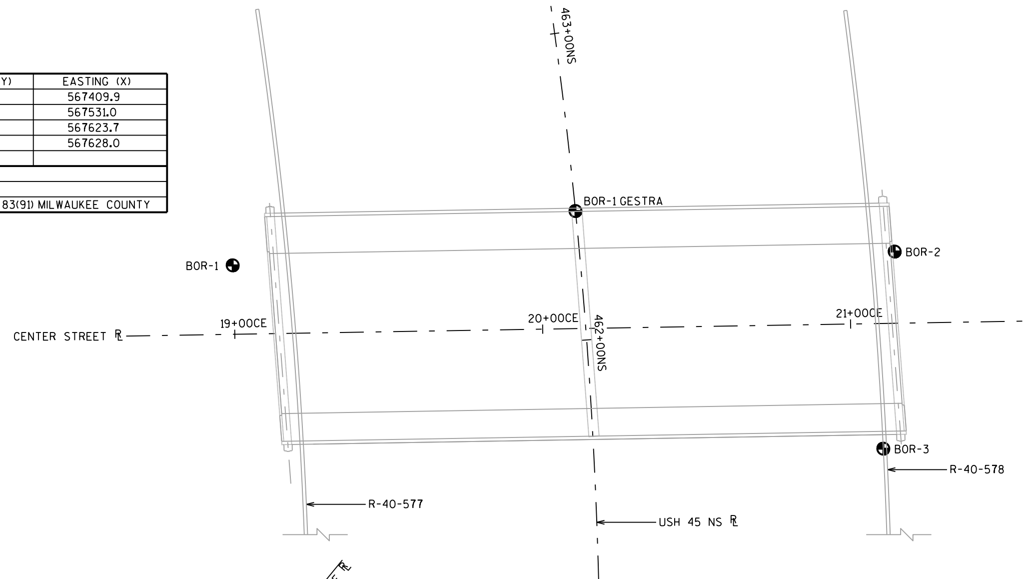
1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2
4. EOB is the end of boring.

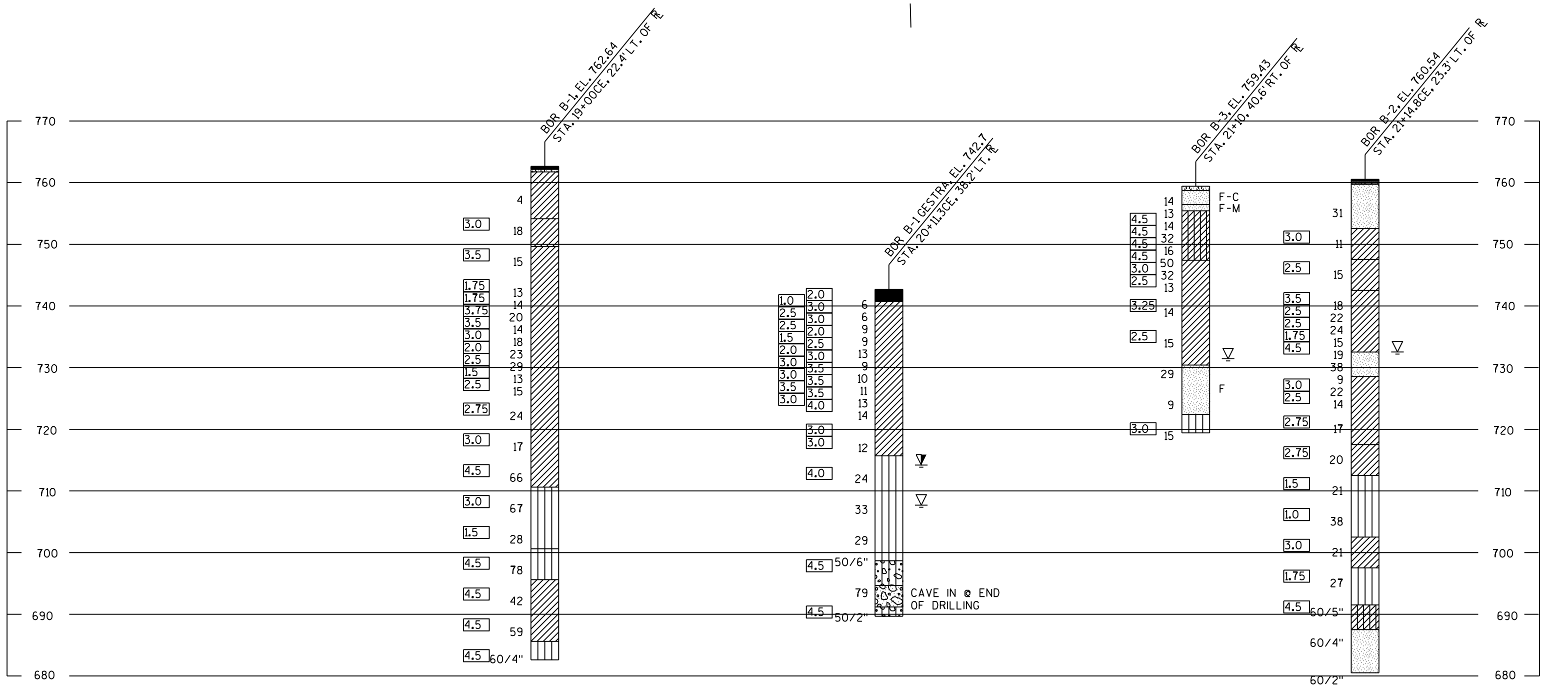
Attachment 2
Bridge Figure

ZOO INTERCHANGE, NORTH LEG
CENTER STREET OVER USH 45

BORING #	DATE COMPLETED	NORTHING (Y)	EASTING (X)
1	11/3/2014	310125.9	567409.9
GESTRA 1	10/16/2014	310131.3	567531.0
2	11/4/2014	310125.5	567623.7
3	11/5/2014	310040.4	567628.0

BORINGS COMPLETED BY: PSI/GESTRA
REPORT COMPLETED BY: WISDOT
ALL COORDINATES REFERENCED TO WCCS NAD 83(91) MILWAUKEE COUNTY





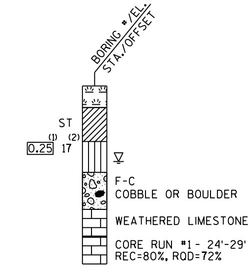
STATE PROJECT NUMBER

1060-33-16

MATERIAL SYMBOLS

ASPHALT — TOPSOIL — PEAT
CONCRETE — FILL — GRAVEL
SAND — CLAY — SILT
BOULDERS OR COBBLES — LIMESTONE — BEDROCK (UNKNOWN)
SHALE — SANDSTONE — IGNEOUS/META

LEGEND OF BORING



(1) UNCONFINED STRENGTH, AS DETERMINED BY A POCKET PENETROMETER (TSF)

(2) UNLESS OTHERWISE, SPECIFIED THE SPT 'N' VALUE IS BASED ON AASHTO T-206, STANDARD PENETRATION TEST. THE SPT 'N' VALUE PRESENTED HAS NOT BEEN CORRECTED FOR OVERBURDEN PRESSURE OR HAMMER EFFICIENCY.

GROUND WATER ELEVATION

- AT TIME OF DRILLING
- END OF DRILLING
- AFTER DRILLING

ABBREVIATIONS

F-FINE M-MEDIUM C-COARSE ST-SHELBY TUBE

SUBSURFACE EXPLORATION FOR FOUNDATION DESIGN AND BIDDERS INFORMATION

BORINGS WERE COMPLETED AT POINTS APPROXIMATELY AS INDICATED ON THIS DRAWING TO OBTAIN INFORMATION CONCERNING THE CHARACTER OF SUBSURFACE MATERIALS FOUND AT THE SITE. BECAUSE THE INVESTIGATED DEPTHS ARE LIMITED AND THE AREA OF THE BORINGS IS VERY SMALL IN RELATION TO THE ENTIRE SITE, THE WISCONSIN DEPARTMENT OF TRANSPORTATION DOES NOT WARRANT SIMILAR SUBSURFACE CONDITIONS BELOW, BETWEEN, OR BEYOND THESE BORINGS. VARIATIONS IN SOIL CONDITIONS SHOULD BE EXPECTED AND FLUCTUATIONS IN GROUNDWATER LEVELS MAY OCCUR.

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-40-880

DRAWN BY PR — PLANS CK'D.

SUBSURFACE EXPLORATION

SHEET

8

SCALE =

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16 WISDOT STRUCTURE ID: B-40-880-2		BORING ID: B-1 PAGE NO: 1 of 4	
WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE: N43° 04.048'	LONGITUDE: W88° 03.229'
ROADWAY NAME: Center Street Over USH 45	DRILLING CONTRACTOR: PSI	DRILLING CONTRACTOR PROJECT NO: 0052853-7	NORTHING:	EASTING:
DATE STARTED: 11/03/14	CREW CHIEF: P. Rotaru	DRILL RIG: Freightliner	COORDINATE SYSTEM: Lat/Long	
DATE COMPLETED: 11/03/14	LOGGED BY: D. Zuydhoek	HOLE SIZE: 10 in	HORIZONTAL DATUM: WGS 1984	VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA	
STATION: 462+35 / OFFSET: 112.5' LT	TOWNSHIP: / RANGE: / SECTION:	1/4 SECTION: / 1/4 1/4 SECTION:	SURFACE ELEVATION: 762.64 ft	

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
				0.5		ASPHALT, (5.5" Thick) 762.1						HSA	
				0.9		BASE COURSE, (5" Thick) 761.7	GPS						
				1–3		CLAY, Fill, Dark Brown, Soft, Trace Sand and Gravel							
SPT 1	4	M	3-2-2-3 (4)	4			CL						
				5–8									
				8.5		754.1							
SPT 2	24	M	5-6-12-17 (18)	9		CLAY, Brown, Very Stiff, Trace Sand and Gravel	CL	3.0					
				10–12									
				13.0		749.6							
SPT 3	24	M	8-8-7-11 (15)	14		CLAY, Gray, Very Stiff, Trace to Few Sand and Gravel		3.5					
				15–18									
SPT 4	24	M	4-5-8-7 (13)	19				1.75					
SPT 5	24	M	4-6-8-8 (14)	21		Stiff	CL	1.75					
SPT 6	24	M	6-9-11-10 (20)	23		Very Stiff		3.75					
SPT 7	24	M	6-6-8-11 (14)	25				3.5					
SPT 8	24	M	7-8-10-11 (18)	27				3.0					
SPT 9	24	M	11-11-12-12 (23)	29				2.0					

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING:	NE	CAVE - IN DEPTH AT COMPLETION:	NMR (WET ☐ DRY ☐)
WATER LEVEL AT COMPLETION:	NMR	CAVE - IN DEPTH AFTER 0 HOURS:	NMR (WET ☐ DRY ☐)

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
SPT 10	24	M	16-15-14-17 (29)	31		CLAY, Gray, Very Stiff, Trace to Few Sand and Gravel		2.5					
				32		Stiff							
SPT 11	24	M	5-6-7-10 (13)	33				1.5					
				34									
SPT 12	24	M	4-7-8-11 (15)	35		Very Stiff		2.5					
				36									
				37									
				38									
SPT 13	24	M	10-12-12-15 (24)	39				2.75					
				40									
				41			CL						
				42									
				43									
SPT 14	24	M	6-7-10-13 (17)	44				3.0					
				45								MR	
				46									
				47									
				48									
SPT 15	24	M	17-33-33-51 (66)	49				4.5					
				50		Hard							
				51									
				52		52.0 — 710.6							
						SILT, Gray, Very Stiff, Trace Sand							
				53									
SPT 16	24	M	13-25-42-60 (67)	54				3.0					
				55									
				56									
				57			ML						
				58									
SPT 17	24	M	8-12-16-18 (28)	59				1.5					
				60		Stiff							
				61									
				62		62.0 — 700.6							
						SILT, Gray, Hard, Some Sand, Trace Gravel							
				63									
SPT 18	15	M	30-43-35-46 (78)	64				4.5					
				65			ML						
				66									
				67		67.0 — 695.6							
						CLAY, Gray, Hard, Little Sand, Trace Gravel							
				68			CL						
SPT			11-20-22-										

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
19	24	M	27 (42)	70		CLAY, Gray, Hard, Little Sand, Trace Gravel		4.5					
				71									
				72									
				73			CL						
SPT 20	24	M	15-23-36-31 (59)	74				4.5					
				75									
				76									
				77		77.0 685.6							
						SILT, Gray, Hard, Some Sand, Trace Gravel							
				78									
SPT 21	8	M	58-60/4"	79			ML	4.5					
				80		80.0 682.6							

End of Boring at 80.0 ft.

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16		BORING ID: B-1 Gestra
	WISDOT STRUCTURE ID: B-40-880		PAGE NO: 1 of 2
WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE: / LONGITUDE:
ROADWAY NAME: Center Street Over USH 45	DRILLING CONTRACTOR: GESTRA	DRILLING CONTRACTOR PROJECT NO:	NORTHING: / EASTING:
DATE STARTED: 10/16/14	CREW CHIEF: A. Woerpel	DRILL RIG: CME-75	COORDINATE SYSTEM: WCCS
DATE COMPLETED: 10/16/14	LOGGED BY: A. Woerpel	HOLE SIZE: 3.25 in	HORIZONTAL DATUM: WCCS Milwaukee / VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA
STATION 462+42 / OFFSET ON R/L	TOWNSHIP: / RANGE: / SECTION:	1/4 SECTION: / 1/4 1/4 SECTION:	SURFACE ELEVATION: 742.7 ft

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
SPT 1	5	M	3-3 (6)	1	Asphalt Concrete (2.0 / 740.7)		2.0				HSA	3 1/4 Hollowstem Auger
SPT 2	10	M	2-3-3-4 (6)	3	Moist Brown Clay with Trace Gravel Trace Sand		1.0 3.0					
SPT 3	22	M	2-3-6-7 (9)	5	Color Change To Gray Moist Clay Trace Gravel		2.5 3.0					
SPT 4	24	M	3-4-5-6 (9)	7			2.5 2.0					
SPT 5	24	M	3-6-7-9 (13)	9			1.5 2.5					
SPT 6	24	M	2-3-6-7 (9)	11			2.0 3.0					
SPT 7	24	M	2-4-6-7 (10)	13			3.0 3.5					
SPT 8	24	M	2-5-6-8 (11)	15	Moist Gray Clay Trace Gravel		3.0 3.5					
SPT 9	24	M	2-5-8-10 (13)	17			3.5 3.5					
SPT 10	24	M	2-5-9-10 (14)	19			3.0 4.0					
				20	Wet Pockets							
SPT 11	18	M	3-5-7 (12)	24	(27.0 / 715.7)		3.0 3.0					
SPT 12	18	M	5-10-14 (24)	29	Moist Gray Silt With Trace Sand		4.0					

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING:	NMR	CAVE - IN DEPTH AT COMPLETION:	NMR
WATER LEVEL AT COMPLETION:	NMR	CAVE - IN DEPTH AFTER 0 HOURS:	NMR

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments		USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
				31		Moist Gray Silt With Trace Sand								
				32										
				33										
SPT 13	18	M	10-14-19 (33)	34										
				35		Wet Silt And Sand Mix								
				36										
				37		Wet Gray Silt								
				38										
SPT 14	18	W	12-13-16 (29)	39										
				40										
				41										
				42										
				43		44.0	698.7							
SPT 15	12	M	20-50	44		Moist Silt With Gravel			4.5					
				45										
				46										
				47		48.0	694.7							
				48		Saturated Gray Sand & Gravel								
SPT 16	12	W	16-35-44 (79)	49										
				50										
				51		51.5	691.2							
				52		Moist Silt With Gravel								
						53.0	689.7							
SPT 17	2	M	50/2"	53		End of Boring at 53.0 ft.			4.5					

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16 WISDOT STRUCTURE ID: B-40-880-3		BORING ID: B-2 PAGE NO: 1 of 4	
WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE: N43° 04.048'	LONGITUDE: W88.03.181'
ROADWAY NAME: Center Street	DRILLING CONTRACTOR: PSI	DRILLING CONTRACTOR PROJECT NO: 0052853-4	NORTHING:	EASTING:
DATE STARTED: 11/04/14	CREW CHIEF: P. Rotaru	DRILL RIG: Freightliner	COORDINATE SYSTEM: Lat/Long	
DATE COMPLETED: 11/04/14	LOGGED BY: D. Zuydhoek	HOLE SIZE: 10 in	HORIZONTAL DATUM: WGS 1984	VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA	
STATION 462+20 / OFFSET 102' RT	TOWNSHIP: / RANGE: / SECTION:	1/4 SECTION: / 1/4 1/4 SECTION:	SURFACE ELEVATION: 760.54 ft	

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	Elev.	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
				0.3	ASPHALT, (4" Thick)	760.2						HSA	
				0.6	CONCRETE, (3" Thick)	759.9	GPS						
				0.8	BASE COURSE, (3" Thick)	759.7							
					SAND, Brown, Dense, Some Gravel								
SPT 1	12	M	17-15-16-10 (31)	3.0–5.0			SP						
				8.0	CLAY, Brown, Very Stiff, Trace Sand and Gravel	752.5							
SPT 2	24	M	9-5-6-8 (11)	8.0–10.0			CL	3.0					
				13.0	CLAY, Brown, Very Stiff, Trace Silt, Sand and Gravel	747.5							
SPT 3	24	M	5-7-8-11 (15)	13.0–15.0			CL	2.5					
				18.0	CLAY, Gray, Very Stiff, Trace Sand and Gravel	742.5							
SPT 4	24	M	6-7-11-13 (18)	18.0–20.0			CL	3.5					
SPT 5	24	M	12-10-12-12 (22)	20.0–22.0				2.5					
SPT 6	24	M	11-13-11-12 (24)	22.0–24.0				2.5					
SPT 7	24	M	4-7-8-11 (15)	24.0–26.0	Stiff			1.75					
SPT 8	18	M	5-6-13-15 (19)	26.0–28.0	Hard			4.5					
				28.0	SAND, Gray, Dense, Little Silt	732.5							
SPT 9	24	W	19-22-16-16 (38)	28.0–30.0			SP						

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING: NMR	CAVE - IN DEPTH AT COMPLETION: NMR	WET ☐ DRY ☐
WATER LEVEL AT COMPLETION: NMR	CAVE - IN DEPTH AFTER 0 HOURS: NMR	WET ☐ DRY ☐

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
SPT 10	24	W	2-3-6-9 (9)	31		SAND, Gray, Dense, Little Silt Loose 32.0 728.5	SP						
SPT 11	24	W	6-9-13-15 (22)	32, 33		CLAY, Gray, Very Stiff, Trace Sand and Gravel		3.0					
SPT 12	24	W	4-6-8-8 (14)	34, 35				2.5					
				36, 37		Little Sand	CL						
SPT 13	24	W	5-6-11-12 (17)	38, 39				2.75					
				40, 41, 42		43.0 717.5							
SPT 14	24	M	7-8-12-12 (20)	43, 44		CLAY, Gray, Very Stiff, Trace Gravel		2.75					
				45, 46, 47		48.0 712.5	CL						
SPT 15	24	W	6-9-12-19 (21)	48, 49		SILT, Gray, Stiff, Trace Sand		1.5					
				50, 51, 52								MR	
SPT 16	18	W	17-18-20-22 (38)	53, 54			ML	1.0					
				55, 56, 57		58.0 702.5							
SPT 17	24	W	5-8-13-16 (21)	58, 59		CLAY, Gray, Very Stiff, Trace Sand and Gravel		3.0					
				60, 61, 62		63.0 697.5	CL						
SPT 18	18	W	10-13-14-27 (27)	63, 64		SILT, Gray, Stiff, Trace Sand		1.75					
				65, 66, 67			ML						
SPT	17	W	37-57-	68		69.0 691.5		4.5					

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
19			60/5"	70, 71, 72		SILTY CLAY, Gray, Hard, Trace Sand and Gravel	CL-ML						
				73		73.0 — 687.5							
SPT 20	12	W	53-48-60/4"	74, 75, 76, 77		SAND, Gray, Very Dense, Some Gravel, Trace Silt	SP						
SPT 21	2	W	60/2"	78, 79									
				80		80.0 — 680.5							

End of Boring at 80.0 ft.

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes

WI Dept. of Transportation 3502 Kinsman Blvd. Madison, WI 53704	WISDOT PROJECT ID: 1060-33-16 WISDOT STRUCTURE ID: R-40-578-3		BORING ID: B-3 PAGE NO: 1 of 2	
WISDOT PROJECT NAME: Center Street over US Highway 45	CONSULTANT: Professional Service Industries, Inc.	CONSULTANT PROJECT NO: 0052853-7	LATITUDE: N43° 04.034'	LONGITUDE: W88° 03.180'
ROADWAY NAME: Center Street Over USH 45	DRILLING CONTRACTOR: PSI	DRILLING CONTRACTOR PROJECT NO: 0052853-4	NORTHING:	EASTING:
DATE STARTED: 11/05/14	CREW CHIEF: M. Ball	DRILL RIG: Diedrich D-50	COORDINATE SYSTEM: Lat/Long	
DATE COMPLETED: 11/05/14	LOGGED BY: D. Zuydhoek	HOLE SIZE: 10 in	HORIZONTAL DATUM: WGS 1984	VERTICAL DATUM: MSL
COUNTY: Milwaukee	LOG QC BY:	HAMMER TYPE: Automatic	STREAMBED ELEVATION: NA	
STATION 461+60 / OFFSET 94' RT	TOWNSHIP: / RANGE: / SECTION:	1/4 SECTION: / 1/4 1/4 SECTION:	SURFACE ELEVATION: 759.43 ft	

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
						0.7 TOPSOIL, (8" Thick) 758.7							
SPT 1	18	D	3-5-9-10 (14)	1		SAND, Brown, Firm, Fine to Coarse, Trace Silt and Gravel	SP					HSA	
SPT 2	24	D	12-7-6-6 (13)	3		3.0 756.4 SAND, Light Brown, Firm, Fine to Medium 4.0 755.4	SP						
SPT 3	24	M	4-6-8-11 (14)	5		SILTY CLAY, Brown, Hard, Trace Sand and Gravel	CL-ML	4.5					
SPT 4	12	M	7-12-20-18 (32)	7				4.5					
SPT 5	24	M	5-6-10-12 (16)	9				4.5					
SPT 6	24	M	12-25-25-23 (50)	11		12.0 747.4		4.5					
SPT 7	24	M	18-15-17-17 (32)	13		CLAY, Gray, Very Stiff, Trace Sand and Gravel	CL	3.0					
SPT 8	24	M	4-6-7-7 (13)	15				2.5					
SPT 9	24	M	5-6-8-10 (14)	19				3.25					
SPT 10	24	M	9-7-8-8 (15)	24				2.5					
SPT 11	24	W	28-16-13-13 (29)	29		29.0 730.4 SAND, Gray, Firm, Fine, Little Silt	SP						

WATER LEVEL & CAVE-IN OBSERVATION DATA

WATER ENCOUNTERED DURING DRILLING: NMR		CAVE - IN DEPTH AT COMPLETION: NMR	WET ☐ DRY ☐
WATER LEVEL AT COMPLETION: NMR		CAVE - IN DEPTH AFTER 0 HOURS: NMR	WET ☐ DRY ☐

NOTES: 1) Stratification lines between soil types represent the approximate boundary; gradual transition between in-situ soil layers should be expected.
2) NE = Not Encountered; NMR = No Measurement Recorded

P:\GINT\WISDOT GINT PROJECTS\GINT_40\B-40-880.GPJ Center Street over US Highway 45 2/11/15

SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	Moisture	BLOW COUNTS (N VALUE)	Depth (ft)	Graphic	Soil / Rock Description and Geological Origin for Each Major Unit / Comments	USCS / AASHTO	Strength Qp (tsf)	Liquid Limit (%)	Plasticity Index (%)	Boulders	Drilling Method	Notes
				31		SAND, Gray, Firm, Fine, Little Silt							
				32									
				33									
SPT 12	24	W	3-3-6-6 (9)	34			SP						
				35		Loose							
				36									
				37		37.0 722.4							
						SILT, Gray, Very Stiff, Little Sand, Trace Clay							
				38									
SPT 13	24	W	4-8-7-8 (15)	39			ML	3.0					
				40		40.0 719.4							

End of Boring at 40.0 ft.

CORRESPONDENCE/MEMORANDUM ___________________State of Wisconsin

DATE: April 10, 2015

TO: Casey Wierzchowski, P.E.
Southeast Region Soils Engineer

FROM: Jeffrey D Horsfall, P.E.
Geotechnical Engineer

SUBJECT: **Site Investigation Report**
Project I.D. 1060-33-16
R-40-0577
Center Street over USH 45
(West Abutment B-40-0880)
Milwaukee County

Attached is the Site Investigation Report for the above project.

Please call if you have any questions.

Attachments

cc: Southeast Region (via e-mail)
Bureau of Structures, Structures Design (via e-submit)
Geotechnical File (original)

Site Investigation Report
Project I.D. 1060-33-16
Structure R-40-0577
Center Street over USH 45
(West Abutment B-40-0880)
Milwaukee County
April 10, 2015

1. GENERAL

The project is a retaining wall located along the west side of USH 45 near Center Street, Milwaukee County. A portion of the proposed retaining wall supports the West Abutment of B-40-0880. Table 1 presents the location of the wall compared to the wall stationing

Table 1: Wall Locations

USH 45 Roadway Station	Wall Station	Description
457+75.0, 92.0' left	10+00.0	Beginning of Wall and supports side slope
463+22.0, 94.0' left	12+33.8	End of Wall and supports side slope

The maximum exposed height is 24.9 feet. The proposed wall type is a MSE wall with precast concrete panels. Aesthetics is a key item to consider in the evaluation of the wall. A portion of the wall is located within a cut section of the roadway. Topography in the general vicinity is urban with a bridge approach located near the wall.

The Southeast Region requested that the Geotechnical Unit evaluate a MSE wall with precast concrete panels. The following report presents the results of the subsurface investigation, the design evaluation, the findings, the conclusions and the recommendations.

2. SUBSURFACE CONDITIONS

Wisconsin Department of Transportation contracted with PSI to completed three borings near the proposed wall. Samples were collected with a method conforming to AASHTO T-206, Standard Penetration Test, using an automatic hammer. The purpose of the borings was to define subsurface soil conditions at this site. Soil textures in the boring logs were field identified by the drillers. Attachment 1 presents tables showing the summaries of subsurface conditions logged in the borings at this site and at the time of drilling for the retaining wall. Attachment 2 presents a figure that illustrates the boring locations and graphical representations of the boring logs. The original borings logs are available at the Central Office Geotechnical Engineering Unit and will be made available upon request.

The following describes the subsurface conditions in the three borings:

0.0 feet to 1.0 foot of pavement structure, overlying
0.0 feet to 7.5 feet of dark brown, soft, clay, trace sand and gravel (fill, B-1), overlying
3.0 feet to 36.5 feet of brown, medium hard to hard, clay, trace sand and gravel, overlying
5.0 feet to 25.0 feet of brown to gray, fine to medium, firm to very dense, sand or silt, trace gravel, overlying
Gray, very hard, silt and clay, little sand, trace gravel

Generally, groundwater was not encountered in the borings at the time of drilling.

3. ANALYSIS ASSUMPTIONS

Chapter 14 of the WisDOT Bridge Manual describe ten different types of retaining structures: reinforced cantilever, gabion, post and panel, sheet pile, modular block gravity, mechanically stabilized earth (MSE) with 4 types of facings, and modular bin and crib walls. Geotechnical Engineering Unit procedures require that the wall alternatives requested by the region be evaluated to determine the feasibility at a particular location, from a geotechnical standpoint.

Table 2 presents the design soil parameters utilized for the analyses, which approximate the conditions at B-7, B-6 and B-1.

Table 2: Soil Parameters

Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)
Granular Backfill Within the wall in the reinforcing zone	30	0	120
Fill Behind and below the reinforcing zone	31	0	120
B-7, 11+00			
Silt, gray, trace sand and gravel (Elevation 745.9 ft – 741.4 ft)	0	4,500	135
Sand, gray, fine to medium (Elevation 741.4 ft – 737.4 ft)	36	0	135
Silt, gray, trace sand, trace clay (Elevation 737.4 ft – 723.4 ft)	0	2,500	125
Silt, gray, trace sand, trace clay (Elevation 723.4 ft – 716.4 ft)	0	4,500	135
B-6, 12+00			
Silt, gray, trace clay, trace sand, trace gravel (Elevation 743.4 ft – 738.4 ft)	0	4,500	135
Sand, gray, fine to medium (Elevation 738.4 ft – 732.4 ft)	32	0	120
Clay, gray, little silt, trace sand, trace gravel (Elevation 732.4 ft – 710.4 ft)	0	3,000	128
Clay, gray, little silt, trace sand, trace gravel (Elevation 710.4 ft – 709.4 ft)	0	4,500	135
B-1, 14+60			
Clay, gray, trace gravel (Elevation 738.6 ft – 733.6 ft)	0	3,000	125
Clay, gray, trace gravel (Elevation 733.6 ft – 729.6 ft)	0	2,500	120
Clay, gray, trace gravel (Elevation 729.6 ft – 717.6 ft)	0	2,000	120
Clay and Silt, gray, trace sand and gravel (Elevation 717.6 ft – 705.6 ft)	0	4,500	135

Table 2: Soil Parameters			
Soil Description	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)
B-1, 14+60 (continued)			
Clay, gray, trace sand (Elevation 705.6 ft – 700.6 ft)	0	2,000	120
Silt, gray, some sand, trace gravel (Elevation 700.6 ft and below)	0	25,000	135

The typical wall section used in the analyses had an **exposed** height that varies from 8.7 feet to 24.9 feet. The following assumptions are also included in the analyses:

1. The slope in front and behind the wall is horizontal.
2. Groundwater was not used in the analyses.
3. The granular backfill is free draining and will not become saturated.
4. The minimum embedment depth is 1.5 feet.
5. A surcharge load of 240 psf is included to model pedestrian and lightweight construction equipment.
6. An additional surcharge load equivalent to the weight of the soil behind the abutment is also included in the design.
7. Global stability factor of safety was determined by the computer program STABLPRO.
8. Bearing resistance is determined by Terzaghi's bearing capacity equation.
9. Settlement of the foundation on cohesionless and cohesive soil is based upon methods described in the FHWA Soils and Foundations Manual.

4. RESULTS OF ANALYSIS

The Geotechnical Unit evaluated a MSE wall with precast concrete facing for the project. The wall was evaluated for sliding, overturning, bearing resistance, global stability and settlement.

Table 3 presents the results of the evaluation and the Capacity to Demand Ratio (CDR).
The exposed wall height examined varied from 8.7 feet to 24.9 feet. The length of reinforcement for the wall is determined by meeting the eccentricity requirements (B/4>e) and a minimum embedment length of 8 feet.

The results of the evaluation indicated that if the sliding and bearing resistance requirements are met, then the eccentricity is also met. The global stability of the wall at the critical location was stable with a CDR of greater than 1.0.

The settlement of the foundation was estimated to be less than 1 inches and should occur within years of loading of the wall. The subsurface soils are relatively uniform; therefore, differential settlement should not be an issue.

Table 3: Results of MSE Wall External Stability Evaluation				
Dimensions				
Wall Height (feet)[1]	10.2	13.2	18.8	26.4
Exposed Wall Height (feet)	8.7	11.7	17.3	24.9
Length of Reinforcement (feet)[3]	8.0	9.2	17.4	18.5
Length of Rein. / Wall Height	NA	0.70	0.93	0.70
Wall Station	11+00.0	12+00.0	14+50.0	14+67.2
Boring Used	B-7	B-6	B-1	B-1
Capacity to Demand Ratio (CDR)[4]				
Sliding (CDR > 1.0)	1.4	1.3	1.0	1.5
Eccentricity (CDR > 1.0)	1.5	1.2	1.0	1.3
Global Stability (CDR > 1.0)	NA	NA	2.1	NA
Bearing Resistance (CDR > 1.0)	2.4	1.8	1.1	1.1
Required Bearing Resistance (psf)	6,000	6,000	7,000	7,000

1. The wall height includes an embedment of 1.5 feet.
2. The wall stability evaluation included a surcharge load that was equal to the weight of the soil behind the abutment.
3. The length of reinforcement is the minimum required length.
4. CDR requirements and load and resistance factors are presented in Chapter 14 of the Bridge Manual.
5. NA not applicable, global slope stability was evaluated at the critical wall location.

5. FINDINGS AND CONCLUSIONS

The following findings and conclusions are based upon the subsurface conditions and the analysis:

1. The following describes the subsurface conditions in the three borings:

   0.0 feet to 1.0 foot of pavement structure, overlying
   0.0 feet to 7.5 feet of dark brown, soft, clay, trace sand and gravel (fill, B-1), overlying
   3.0 feet to 36.5 feet of brown, medium hard to hard, clay, trace sand and gravel, overlying
   5.0 feet to 25.0 feet of brown to gray, fine to medium, firm to very dense, sand or silt, trace gravel, overlying
   Gray, very hard, silt and clay, little sand, trace gravel

2. The groundwater was not encountered in the investigation.

3. Table 3 presents the results of the external stability evaluation and shows that if the sliding and bearing resistance requirements are satisfied, then the eccentricity and global stability will also be satisfied.

4. Settlement of the foundation was estimated to be less than 2 inches and should occur within months of loading of the wall. The subsurface soils are relatively uniform; therefore, differential settlement should not be an issue.

6. RECOMMENDATIONS

The following recommendations are based upon the findings and conclusions:

1. The MSE wall with precast concrete panels will achieve the external stability factors of safety if the sliding and bearing resistance requirements are met. Table 3 presents the minimum length of the reinforcement at the locations evaluated. In the area of the wall that supports the abutment, the ratio of length of reinforcement to total height of wall should be increased from 0.70 to 0.93.

2. The contractor should remove 6-inches of topsoil and silt and clay below the reinforcing zone and replace with granular fill in the areas that the topsoil and silt and clay are encountered.

3. The backfill behind the MSE wall with precast concrete facing should be granular and free draining.

4. The Southeast Region soils engineer should review the fill subsurface conditions prior to construction of the wall.

Attachment 1
Tables of Subsurface Conditions

Subsurface Conditions: R-40-0577							
B-7 Station 458+75 85.5 feet left of USH 45 RL				B-6 Station 459+75 85.5 feet left of USH 45 RL			
Estimated Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**	**Estimated Top of Soil Layer Elevation (feet)**	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
751.4	Clay, brown, trace sand and gravel Qp=3.5	9	20	749.4	Clay, brown, trace sand and gravel Qp=2.25 – 2.5	7,12, 8	16,22, 13
748.4	Sand, brown, fine to medium, trace clay	18	33	743.9	Silt, gray, trace clay, trace sand, trace gravel Qp=4.5 – 4.5+	42,26	63,36
747.4	Silt, gray, trace sand and gravel Qp=3.0 – 4.5+	36,56, 62	58,82, 85	738.4	Sand, gray, fine to medium	12,31, 26	16,39, 31
741.4	Sand, gray, fine to medium	55,47	71,57	732.4	Clay, gray, little silt, trace sand, trace gravel Qp=3.25 – 4.5	23,17, 15,18	25,17, 14,16
737.4	Silt, gray, trace sand, trace clay Qp=2.5 – 4.5+	18,25, 18	21,27, 18	710.4	Clay, gray, little silt, trace sand, trace gravel Qp=3.5	43	35
723.4	Silt, gray, trace sand, trace clay Qp=3.5	108, 60/4”	100, 51/4”	709.4	EOB		
716.4	EOB						

1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring.
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2.
4. EOB is the end of boring.

Subsurface Conditions: R-40-0577			
B-1 Station 462+35.0 112.5 feet left of USH 45 RL			
Estimated Top of Soil Layer Elevation (feet)	**Soil Description**	**SPT Blow count**	**Corr. SPT Blow count**
762.6	Pavement Structure		
761.6	Clay, dark brown, trace sand and gravel (fill)	4	7
754.1	Clay, brown, some silt, trace sand and gravel Qp=3.0	18	25
749.6	Clay, gray, trace gravel Qp=1.75 – 3.5	15,13,14	18,14,15
739.6	Clay, gray, trace gravel Qp=3.0 – 3.75	20,14,18	21,14,17
733.6	Clay, gray, trace gravel Qp=2.0 – 2.5	23,29	22,26
729.6	Clay, gray, trace gravel Qp=1.5 – 3.0	13,15,24,17	12,13,20,13
717.6	Clay and Silt, gray, trace sand and gravel Qp=3.0 - 4.5+	66,67	49,47
705.6	Silt, gray, trace sand Qp=1.5	28	18
700.6	Silt, gray, some sand, trace gravel Qp=4.5+	78,42,59, 60/4”	49,25,34, 33/4”
682.6	EOB		

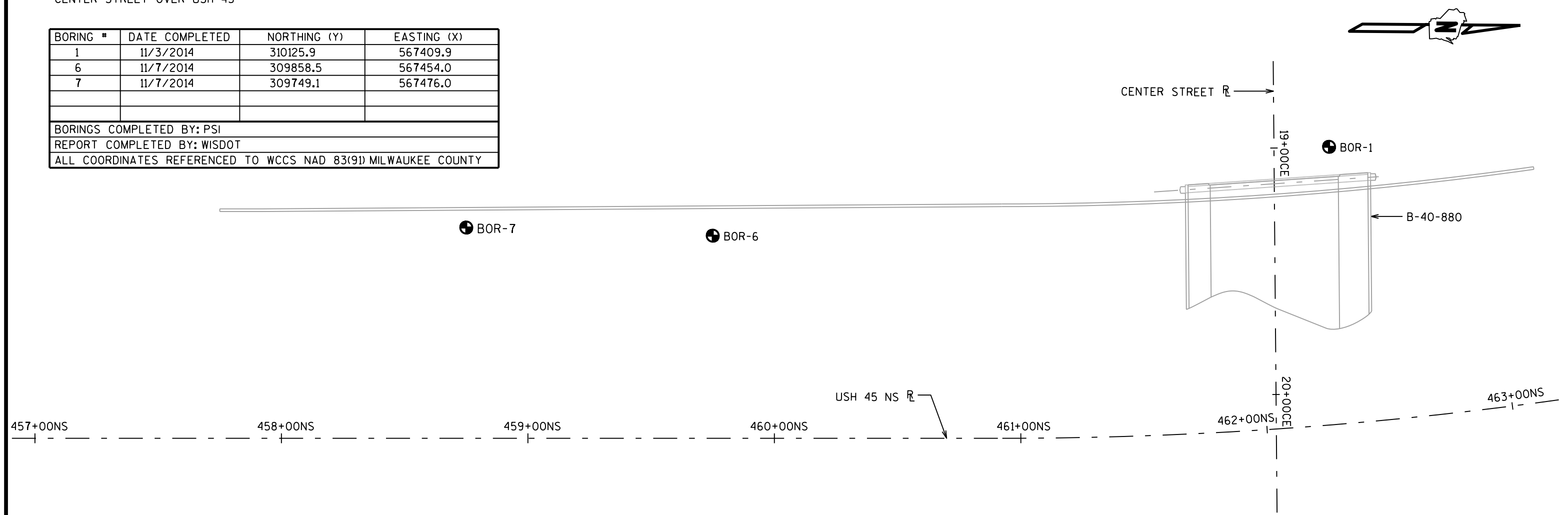
1. Blow counts are corrected for SPT hammer efficiency and overburden pressure.
2. First elevation is the surface elevation for the boring.
3. Qp = Unconfined compression strength as determined by a pocket penetrometer, tons/ft^2.
4. EOB is the end of boring.

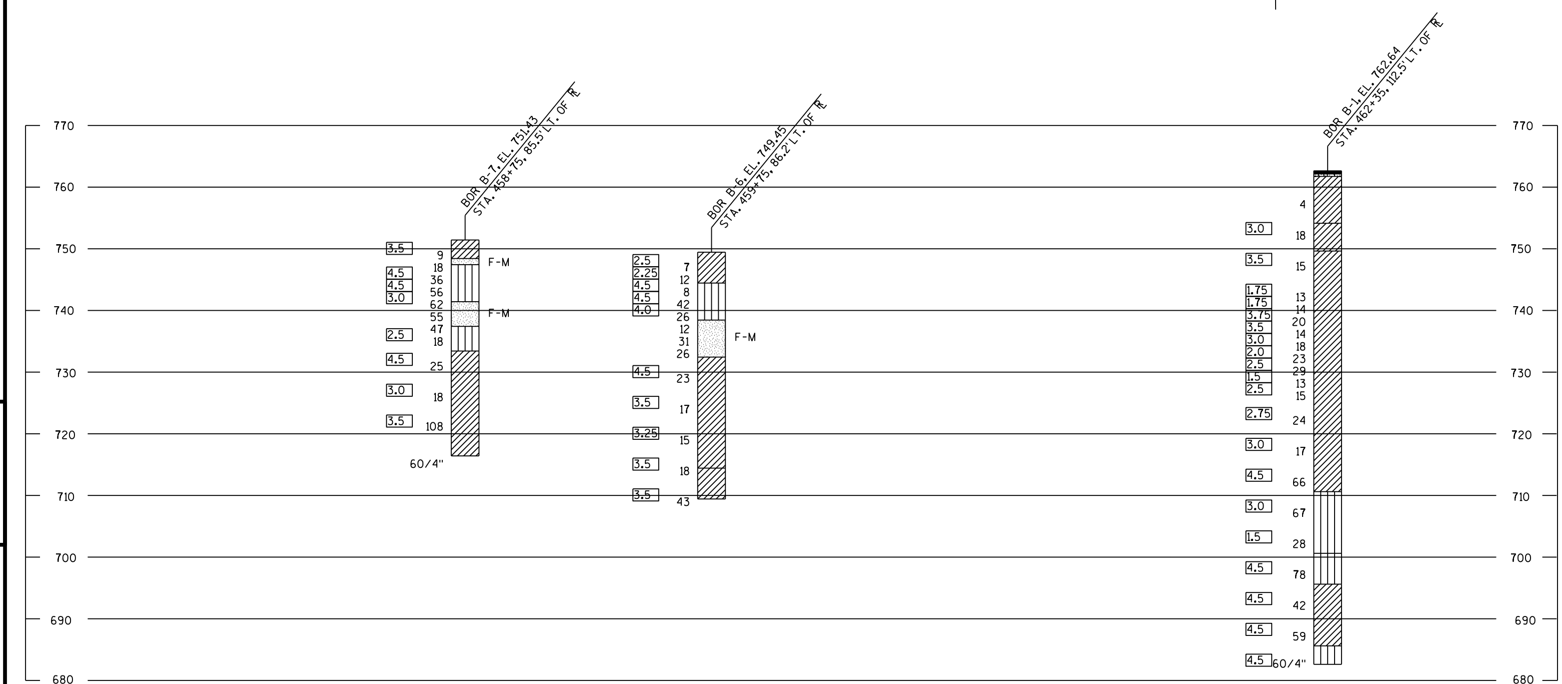
Attachment 2
Wall Figure

ZOO INTERCHANGE, NORTH LEG
CENTER STREET OVER USH 45

BORING #	DATE COMPLETED	NORTHING (Y)	EASTING (X)
1	11/3/2014	310125.9	567409.9
6	11/7/2014	309858.5	567454.0
7	11/7/2014	309749.1	567476.0

BORINGS COMPLETED BY: PSI
REPORT COMPLETED BY: WISDOT
ALL COORDINATES REFERENCED TO WCCS NAD 83(91) MILWAUKEE COUNTY





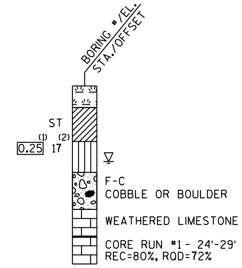
STATE PROJECT NUMBER

1060-33-16

MATERIAL SYMBOLS

ASPHALT · TOPSOIL · PEAT
CONCRETE · FILL · GRAVEL
SAND · CLAY · SILT
BOULDERS OR COBBLES · LIMESTONE · BEDROCK (UNKNOWN)
SHALE · SANDSTONE · IGNEOUS/ META

LEGEND OF BORING



(1) UNCONFINED STRENGTH, AS DETERMINED BY A POCKET PENETROMETER (TSF)

(2) UNLESS OTHERWISE SPECIFIED THE SPT 'N' VALUE IS BASED ON AASHTO T-206, STANDARD PENETRATION TEST. THE SPT 'N' VALUE PRESENTED HAS NOT BEEN CORRECTED FOR OVERBURDEN PRESSURE OR HAMMER EFFICIENCY.

GROUND WATER ELEVATION

- AT TIME OF DRILLING
- END OF DRILLING
- AFTER DRILLING

ABBREVIATIONS

F-FINE · M-MEDIUM · C-COARSE · ST-SHELBY TUBE

SUBSURFACE EXPLORATION FOR FOUNDATION DESIGN AND BIDDERS INFORMATION

BORINGS WERE COMPLETED AT POINTS APPROXIMATELY AS INDICATED ON THIS DRAWING TO OBTAIN INFORMATION CONCERNING THE CHARACTER OF SUBSURFACE MATERIALS FOUND AT THE SITE. BECAUSE THE INVESTIGATED DEPTHS ARE LIMITED AND THE AREA OF THE BORINGS IS VERY SMALL IN RELATION TO THE ENTIRE SITE, THE WISCONSIN DEPARTMENT OF TRANSPORTATION DOES NOT WARRANT SIMILAR SUBSURFACE CONDITIONS BELOW, BETWEEN, OR BEYOND THESE BORINGS. VARIATIONS IN SOIL CONDITIONS SHOULD BE EXPECTED AND FLUCTUATIONS IN GROUNDWATER LEVELS MAY OCCUR.

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE R-40-577

DRAWN BY PR · PLANS CK'D.

SUBSURFACE EXPLORATION

SHEET

SCALE =

8

8

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11.1 General

11.1.1 Overall Design Process

The overall foundation support design process requires an iterative collaboration to provide cost-effective constructible substructures. Input is required from multiple disciplines including, but not limited to, structural, geotechnical and design. For a typical bridge design, the following four steps are required (see 6.2):

1. Structure Survey Report (SSR) – This design step results in a very preliminary evaluation of the structure type and approximate location of substructure units, including a preliminary layout plan.

2. Site Investigation Report – Based on the Structure Survey Report, a Geotechnical Investigation (see Chapter 10 – Geotechnical Investigation) is required, including test borings to determine foundation requirements. A hydraulic analysis is also performed at this time, if required, to assess scour potential and maximum scour depth. The Site Investigation Report and Subsurface Exploration Drawing are used to identify known constraints that would affect the foundations in regard to type, location or size and includes foundation recommendations to support detailed structural design. Certain structure sites/types may require the preliminary structure plans (Step 3) prior to initiating the geotechnical site investigation. One example of this is a multi-span structure over water. See 6.2 for more information.

3. Preliminary Structure Plans – This design step involves preparation of a general plan, elevation, span arrangement, typical section and cost estimate for the new bridge structure. The Site Investigation Report is used to identify possible poor foundation conditions and may require modification of the structure geometry and span arrangement. This step may require additional geotechnical input, especially if substructure locations must be changed.

4. Final Contract Plans for Structures – This design step culminates in final plans, details, special provisions and cost estimates for construction. The Subsurface Exploration sheet(s) are part of the Final Contract Plans. Unless design changes are required at this step, additional geotechnical input is not typically required to prepare foundation details for the Final Contract Plans.

11.1.2 Foundation Type Selection

The following items need to be assessed to select site-specific foundation types:

- Magnitude and direction of loads.
- Depth to suitable bearing material.
- Potential for liquefaction, undermining or scour.
- Frost potential.

11.3.1.12.2 Concrete Piles

The three principal types of concrete pile are cast-in-place (CIP), precast reinforced and prestressed reinforced. CIP concrete pile types include piles cast in driven steel shells that remain in-place, and piles cast in unlined drilled holes or shafts. Driven-type concrete pile is discussed below in this section. Concrete pile cast in drilled holes is discussed later in this chapter and include drilled shafts (11.3.2), micropiles (11.3.3), and augered cast-in-place piles (11.3.4).

Depending on the type of concrete pile selected and the foundation conditions, the load-carrying capacity of the pile can be developed by shaft resistance, point resistance or a combination of both. Generally, driven concrete pile is employed as a displacement type pile.

When embedded in the earth, plain or reinforced concrete pile is generally not vulnerable to deterioration. The water table does not affect pile durability provided the concentration level is not excessive for acidity, alkalinity or chemical salt. Concrete pile that extends above the water surface is subject to abrasion damage from floating objects, ice debris and suspended solids. Deterioration can also result from frost action, particularly in the splash zone and from concrete spalling due to internal corrosion of the reinforcement steel. Generally, concrete spalls are a concern for reinforced concrete pile more than prestressed pile because of micro-cracks due to shrinkage, handling, placement and loading. Prestressing reduces crack width. Concrete durability increases with a corresponding reduction in aggregate porosity and water/cement ratio. WisDOT does not currently use prestressed reinforced concrete pile.

11.3.1.12.2.1 Driven Cast-In-Place Concrete Piles

Driven cast-in-place (CIP) concrete piles are formed by pouring concrete into a thin-walled closed-end steel shell which has been previously driven into the ground. A flat, oversize plate is typically welded to the bottom of the steel shell. Steel shells are driven either with or without a mandrel, depending on the wall thickness of the steel shell and the shell strength that is required to resist driving stress. Piling in Wisconsin is typically driven without the use of a mandrel. The minimum thickness of the steel shell should be that required for pile reinforcement and to resist driving stress. The Contractor may elect to furnish steel shells with greater thickness to permit their choice of driving equipment. A thin-walled shell must be carefully evaluated so that it does not collapse from soil pressure or deform from adjacent pile driving. Deformities or distortions in the pile shell could constrict the flow of concrete into the pile and produce voids or necking that reduce pile capacity. It is standard construction practice to inspect the open shell prior to concrete placement. Care must be exercised to avoid intermittent voids over the pile length during concrete placement.

Driven CIP concrete piles are considered a displacement-type pile, because the majority of the applied load is usually supported by shaft resistance. This pile type is frequently employed in slow flowing streams and areas requiring pile lengths of 50 to 120 feet. Driven CIP pile is generally selected over timber pile because of the availability of different diameters and wall thicknesses, the ability to adjust driven lengths and the ability to achieve greater resistances.

Driven CIP concrete piles may have a uniform cross section or may be tapered. The minimum cross-sectional area is required to be 100 and 50 square inches at the pile butt and tip, respectively. The Department has only used a limited number of tapered CIP piles and has experienced some driving problems with them.

For consistency with WisDOT design practice, the steel shell is ignored when computing the axial structural resistance of driven CIP concrete pile that is symmetrical about both principal axes. This nominal (ultimate) axial structural resistance capacity is computed using the following equation, neglecting the contribution of the steel shell to resist compression: **LRFD [Equation 5.7.4.4-3]**.

$$P_u \le P_r = \phi P_n$$

Where:

$$P_n = 0.80(k_C \cdot f'_c \cdot (A_g - A_{st})) + f_y \cdot A_{st}$$

Where:

P_u	=	Factored axial force effect (kips)
P_r	=	Factored axial resistance without flexure (kips)
ϕ	=	Resistance factor
P_n	=	Nominal axial resistance without flexure (kips)
A_g	=	Gross area of concrete pile section (inches^2)
A_{st}	=	Total area of longitudinal reinforcement (inches^2)
k_c	=	Ratio of max. concrete compressive stress to specified compressive strength of concrete; k_C = 0.85 (for $f'_c \le$ 10.0 ksi)
f_y	=	Specified yield strength of reinforcement (ksi)
f'_c	=	Concrete compressive strength (ksi)

For cast-in-place concrete piles with steel shell and no steel reinforcement bars, A_{st} equals zero and the above equation reduces to the following.

$$P_n = 0.68 f'_c A_g$$

A resistance factor, ϕ, of 0.75 is used to compute the factored structural axial resistance capacity, as specified in **LRFD [5.5.4.2.1]**. For CIP piling there are no reinforcing ties, however the steel shell acts to confine concrete similar to ties.

$$P_r = 0.51 f_c A_g$$

For piles subject to large lateral loads, the structural pile capacity must also be checked for shear and combined stress against flexure and compression.

Piles subject to uplift must also be checked for tension resistance.

A concrete compressive strength of 4 ksi is the minimum value required by specification, while a value of 3.5 ksi is used in the structural design computations. Pile capacities are maximums, based on an assumed concrete compressive strength of 3.5 ksi. The concrete compressive strength of 3.5 ksi is based on construction difficulties and unknowns of placement. The Geotechnical Site Investigation Report must be used as a guide in determining the nominal geotechnical resistance for the pile.

Any structural strength contribution associated with the steel shell is neglected in driven CIP concrete pile design. Therefore, environmentally corrosive sites do not affect driven CIP concrete pile designs. An exception is that CIP should not be used for exposed pile bents in corrosive environments as shown in the *Facilities Development Manual,* Procedure 13-1-15.

Based on the above equation, current WisDOT practice is to design driven cast-in-place concrete piles for factored (ultimate structural) axial compression resistances as shown in Table 11.3-5. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.** The minimum shell thickness is 0.219 inches for straight steel tube and 0.1793 inches for fluted steel shells, unless otherwise noted in the Geotechnical Site Investigation Report and stated in the project plans. Exposed piling (e.g. open pile bents) should not be less than 12 inches in diameter.

When cobbles or other difficult driving conditions are present, the minimum wall thickness for steel shells of driven cast-in-place concrete piles should be increased to 0.25 inches or thicker to facilitate driving without damaging the pile. A drivability analysis should be completed in design, to determine the required wall thickness based on site conditions and an assumed driving equipment.

Driven cast-in-place concrete pile is generally the most favorable displacement pile type since inspection of the steel shell is possible prior to concrete placement and more reliable control of concrete placement is attainable.

11.3.1.12.2.2 Precast Concrete Piles

Precast concrete pile can be divided into two primary types – reinforced concrete piles and prestressed concrete piles. These piles have parallel or tapered sides and are usually of rectangular or round cross section. Since the piles are usually cast in a horizontal position, the round cross section is not common because of the difficulty involved in filling a horizontal cylindrical form. Because of the somewhat variable subsurface conditions in Wisconsin and the need for variable length piles, these piles are currently not used in Wisconsin.

11.3.1.12.3 Steel Piles

Steel pile generally consist of either H-pile or pipe pile types. Both open-end and closed-end pipe pile are used. Pipe piles may be left open or filled with concrete, and can also have a structural shape or reinforcement steel inserted into the concrete. Open-end pipe pile can be socketed into bedrock with preboring.

Steel pile is typically top driven at the pile butt. However, closed-end pipe pile can also be bottom driven with a mandrel. Mandrels are generally not used in Wisconsin.

Steel pile can be used in friction, point-bearing, a combination of both, or rock-socketed piles. One advantage of steel pile is the ease of splicing or cutting to accommodate differing final constructed lengths.

Steel pile should not be used for exposed pile bents in corrosive environments as show in the *Facilities Development Manual, Procedure 13.1.15*.

The nominal (ultimate) axial structural compressive resistance of steel piles is designed in accordance with **LRFD [10.7.3.13.1]** as either non-composite or composite sections. Composite sections include concrete-filled pipe pile and steel pile that is encased in concrete. The nominal structural compressive resistance for non-composite and composite steel pile is further specified in **LRFD [6.9.4 and 6.9.5]**, respectively. The effective length of horizontally unsupported steel pile is determined in accordance with **LRFD [10.7.3.13.4]**. Resistance factors for the structural compression limit state are specified in **LRFD [6.5.4.2]**.

WisDOT policy item:

Specify a yield strength of 50 ksi for steel H-piles. Although 50 ksi is specified, the structural pile design shall use a yield strength of 36 ksi. The specified yield strength of 50 ksi may be used when performing drivability analyses. For steel pipe piles, 35 ksi shall be used for pile design and drivability analyses.

11.3.1.12.3.1 H-Piles

Steel piles are generally used for point-bearing piles and typically employ what is known as the HP-section (often called H-piles for brevity). Steel H-piles are rolled sections with wide flanges such that the depth of the section and the width of the flanges are approximately equal. The cross-sectional area and volume displacement are relatively small and as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered “non-displacement” piling.

H-piles are available in many sizes and lengths. Unspliced pile lengths up to 140 feet and spliced pile lengths up to 230 feet have been driven. Typical pile lengths range from 40 to 120 feet. Common H-pile sizes vary between 10 and 14 inches.

The current WisDOT practice is to design driven H-piles for the factored (ultimate structural) axial compression resistance as shown in Table 11.3-5. These values are based on $\phi_c = 0.5$ for severe driving conditions **LRFD [6.5.4.2]**. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.**

Since granular soil is largely incompressible, the principal action at the tip of the pile is lateral displacement of soil particles. Although it is an accepted fact that steel piles develop extremely high loads per pile when driven to point-bearing on rock, some misconceptions still remain that H-piles cannot function as friction piles. Load tests indicate that steel H-piles can function quite satisfactorily as friction piles in sand, sand-clay, silt-and-sand or hard clay. However, they are not as efficient as displacement piles in these conditions and typically drive to greater depths. The surface area for pile frictional computations is considered to be the projected “box area” of the H-pile, and not the actual steel surface area.

Clay is compressible to a far greater degree than sand or gravel. As the solid particles are pressed into closer contact with each other and water is squeezed out of the voids, only small frictional resistance to driving is generated because of the lubricating action of the free water. However, after driving is completed, the lateral pressure against the pile increases due to dissipation of the pore water pressures. This causes the fine clay particles to increase adherence to the comparatively rough surface of the pile. Load is transferred from the pile to the soil by the resulting strong adhesive bond. In many types of clay, this bond is stronger than the shearing resistance of the soil.

In hard, stiff clays containing a low percentage of voids and pore water, the compressibility is small. As a result, the amount of displacement and compression required to develop the pile’s full capacity are correspondingly small. As an H-pile is driven into stiff clay, the soil trapped between the flanges and web usually becomes very hard due to the compression and is carried down with it. This trapped soil acts as a plug and the pile can also act as a displacement pile.

In cases where loose soil is encountered, considerably longer point-bearing steel piles are required to carry the same load as relatively short displacement-type piles. This is because a displacement-type pile, with its larger cross section, produces more compaction as it is driven through materials such as soft clays or loose organic silts. H-piles are not typically used in exposed pile bents due to concerns with debris catchment.

Pipe Piles

Pipe piles consist of seamless, welded or spiral welded steel pipes in diameters ranging from 7-3/4 to 24 inches. Other sizes are available, but they are not commonly used. Typical wall thicknesses range from 0.375-inch to 0.75-inch, with wall thicknesses of up to 1.5 inches possible. Pipe piles should be specified by grade with reference to ASTM A 252.

Pipe piles may be driven either open or closed end. If the end bearing capacity from the full pile toe area is required, the pile toe should be closed with a flat plate or a conical tip.

11.3.1.12.3.2 Oil Field Piles

The oil industry uses a very high quality pipe in their drilling operations. Every piece is tested for conformance to their standards. Oil field pipe is accepted as a point-bearing alternative to

HP piling, provided the material in the pipe meets the requirements of ASTM A 252, Grade 3, with a minimum tensile strength of 120 ksi or a Brinell Hardness Number (BHN) of 240, a minimum outside diameter of 7-3/4 inches and a minimum wall thickness of 0.375-inch. The weight and area of the pipe shall be approximately the same as the HP piling it replaces. Sufficient bending strength shall be provided if the oil field pipe is replacing HP piling in a pile bent. Oil field pipe is driven open-ended and not filled with concrete. The availability of this pile type varies and is subject to changes in the oil industry.

11.3.1.12.4 Pile Bents

See 13.1 for criteria to use pile bents at stream crossings. When pile bents fail to meet these criteria, pile-encased pier bents should be considered. To improve debris flow, round piles are generally selected for exposed bents. Round or H-piles can be used for encased bents.

11.3.1.13 Tolerable Movement of Substructures Founded on Driven Piles

WisDOT policy item:

For design of new bridge structures founded on driven piles, limit the horizontal movement at the top of the foundation unit to 0.5 inch or less at the service limit state.

11.3.1.14 Resistance Factors

The nominal (ultimate) geotechnical resistance capacity of the pile should be based on the type, depth and condition of subsurface material and ground water conditions reported in the Geotechnical Site Investigation Report, as well as the method of analysis used to determine pile resistance. Resistance factors to compute the factored geotechnical resistance are presented in **LRFD [Table 10.5.5.2.3-1]** and are selected based on the method used to determine the nominal (ultimate) axial compression resistance. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal pile resistance. When construction controls, are used to improve the reliability of capacity prediction (such as pile driving analyzer or static load tests), the resistance factors used during final design should be increased in accordance with **LRFD [Table 10.5.5.2.3-1]** to reflect planned construction monitoring.

WisDOT exception to AASHTO:

WisDOT requires at least four (4) piles per group to support each substructure unit, including each column for multi-column bents. WisDOT does not reduce geotechnical resistance factors to satisfy redundancy requirements to determine axial pile resistance. Hence, redundancy resistance factors in **LRFD [10.5.5.2.3]** are not applicable to WisDOT structures. This exception applies to typical CIP concrete pile and H-pile foundations. Non-typical foundations (such as drilled shafts) shall be investigated individually.

No guidance regarding the structural design of non-redundant driven pile groups is currently included in *AASHTO LRFD*. Since WisDOT requires a minimum of 4 piles per substructure unit, structural design should be based on a load modifier, η, of 1.0. Further description of load modifiers is presented in **LRFD [1.3.4]**.

The following geotechnical resistance factors apply to the majority of the Wisconsin bridges that are founded on driven pile. On the majority of WisDOT projects, wave equation analysis and dynamic monitoring are not used to set driving criteria. This equates to typical resistance factors of 0.35 to 0.45 for pile design. A summary of resistance factors is presented in Table 11.3-1, which are generally used for geotechnical design on WisDOT projects.

Condition/Resistance Determination Method			Resistance Factor
Static Analysis – Used in Design Phase	Nominal Resistance of Single Pile in Axial Compression, φ_{stat}	Skin Friction and End Bearing in Clay and Mixed Soil Alpha Method	0.35
		Skin Friction and End Bearing in Sand Nordlund/Thurman Method	0.45
		Point Bearing in Rock	0.45
	Block Failure, φ_{bl}	Clay	0.60
	Uplift Resistance of Single Pile, φ_{up}	Clay and Mixed Soil Alpha Method	0.25
		Sand Nordlund Method	0.35
	Horizontal Resistance of Single Pile or Pile Group	All Soil Types and Rock	1.0
Nominal Resistance of Single Pile in Axial Compression – Dynamic Analysis – for the Hammer and Pile Driving System Actually - used During Construction for Pile Installation, φ_{dyn}		FHWA-modified Gates dynamic pile driving formula (end of drive condition only)	0.50
		Wave equation analysis, without pile dynamic measurements or load test, at end of drive condition only	0.50
		Driving criteria established by dynamic test with signal matching at beginning of redrive conditions only of at least one production pile per substructure, but no less than the number of tests per site provided in **LRFD [Table 10.5.5.2.3-3]**; quality control of remaining piles by calibrated wave equation and/or dynamic testing	0.65

<u>Table 11.3-1</u>
Geotechnical Resistance Factors for Driven Pile

Resistance factors for structural design of piles are based on the material used, and are presented in the following sections of *AASHTO LRFD*:

- Concrete piles – **LRFD [5.5.4.2.1]**
- Steel piles – **LRFD [6.5.4.2]**

11.3.1.15 Bearing Resistance

A pile foundation transfers load into the underlying strata by either shaft resistance, point resistance or a combination of both. Any driven pile will develop some amount of both shaft and point resistance. However, a pile that receives the majority of its support capacity by friction or adhesion from the soil along its shaft is referred to as a friction pile, whereas a pile that receives the majority of its support from the resistance of the soil near its tip is a point resistance (end bearing) pile.

The design pile capacity is the maximum load the pile can support without exceeding the allowable movement criteria. When considering design capacity, one of two items may govern the design – the nominal (ultimate) geotechnical resistance capacity or the structural resistance capacity of the pile section. This section focuses primarily on the geotechnical resistance capacity of a pile.

The factored load that is applied to a single pile is carried jointly by the soil beneath the tip of the pile and by the soil around the shaft. The total factored load is not permitted to exceed the factored resistance of the pile foundation for each limit state in accordance with **LRFD [1.3.2.1 and 10.7.3.8.6]**. The factored bearing resistance, or pile capacity, of a pile is computed as follows:

$$\sum \eta_i \gamma_i Q_i \leq R_r = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s$$

Where:

η_i	=	Load modifier
γ_i	=	Load factor
Q_i	=	Force effect (tons)
R_r	=	Factored bearing resistance of pile (tons)
R_n	=	Nominal resistance (tons)
R_p	=	Nominal point resistance of pile (tons)
R_s	=	Nominal shaft resistance of pile (tons)
φ	=	Resistance factor
φ_{stat}	=	Resistance factor for driven pile, static analysis method

This equation is illustrated in Figure 11.3-1.

Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Axial Compression Resistance (Pn) (tons) (2)(3)(6)	Resistance Factor (ϕ)	Factored Axial Compression Resistance (Pr) (tons) (4)	Resistance Factor φ_{dyn}	Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles							
10 ¾“	0.219	83.5	99.4	0.75	55(8)	0.5	110
10 ¾“	0.250	82.5	98.2	0.75	65(8)	0.5	130
10 ¾“	0.365	78.9	93.8	0.75	75	0.5	150
10 ¾“	0.500	74.7	88.8	0.75	75(9)	0.5	150
12 ¾”	0.250	118.0	140.4	0.75	80(8)	0.5	160
12 ¾”	0.375	113.1	134.6	0.75	105	0.5	210
12 ¾”	0.500	108.4	129.0	0.75	105(9)	0.5	210
14”	0.250	143.1	170.3	0.75	85(8)	0.5	170
14”	0.375	137.9	164.1	0.75	120	0.5	240
14”	0.500	132.7	158.0	0.75	120(9)	0.5	240
16”	0.375	182.6	217.3	0.75	145(8)	0.5	290
16”	0.500	176.7	210.3	0.75	160	0.5	320
H-Piles							
10 x 42	NA(1)	12.4	310.0	0.50	90(10)	0.5	180
12 x 53	NA(1)	15.5	387.5	0.50	110(10)	0.5	220
14 x 73	NA(1)	21.4	535.0	0.50	125(10)	0.5	250

Table 11.3-5
Typical Pile Resistance Values

Notes

1. NA – not applicable

2. For CIP Piles: Pn = 0.8 (k_C * f’c * Ag + fy * As) **LRFD [5.7.4.4-3]**. k_C = 0.85 (for $f'_C \leq$ 10.0 ksi). Neglecting the steel shell, equation reduces to 0.68 * f’c * Ag.

   f’c = compressive strength of concrete = 3,500 psi

3. For H-Piles: Pn = (0.66^{λ} * Fe * As) **LRFD [6.9.5.1-1]** (λ = 0 for piles embedded in the ground below the substructure, i.e. no unsupported lengths)

   Fe = fy = yield strength of steel = 50,000 psi

4. $Pr = \phi * Pn$

   $\phi = 0.75$ (**LRFD [5.5.4.2.1]** for axial compression concrete)

   $\phi = 0.50$ (**LRFD [6.5.4.2]** for axial steel, for difficult driving conditions)

5. The Required Driving Resistance is the lesser of the following:

   - $Rn_{dyn} = Pr / \varphi_{dyn}$

     φ_{dyn} = 0.5 for construction driving criteria using modified Gates dynamic formula)

   - The maximum allowable driving stress based on 90 percent of the specified yield stress = 35,000 psi for CIP piles and 50,000 psi for H-Piles

6. Values for Axial Compression Resistance are calculated assuming the pile is fully supported. Piling not in the ground acts as an unbraced column. Calculations verify that the pile values given in Table 11.3-5 are valid for open pile bents within the limitations described in 13.2.2. Cases of excessive scour require the piling to be analyzed as unbraced columns above the point of streambed fixity.

7. If less than the maximum axial resistance, P_r, is required by design, state only the required corresponding driving resistance on the plans.

8. The Factored Axial Compression Resistance is controlled by the maximum allowable driving resistance based on 90 percent of the specified yield stress of steel rather than concrete capacity.

9. Values were rounded up to the value above so as to not penalize the capacity of the thicker walled pile of the same diameter. (Wisconsin is conservative in not considering the pile shell in the calculation of the Factored Axial Compression Resistance.)

10. Pr values given for H-Piles are representative of past Departmental experience (rather than Pn x ø) and are used to avoid problems associated with overstressing during driving. These Pr values result in driving stresses much less than 90 percent (46%-58%) of the specified yield stress. If other H-Piles are utilized that are not shown in the table, driving stresses should be held to approximately this same range.

11.3.1.18 Construction Considerations

Construction considerations generally include selection of pile hammers, use of driving formulas and installation of test piles, when appropriate, as described below.

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should be used for intermediate abutment heights. The load factors for both vertical and horizontal components of live load surcharge are as specified in **LRFD [Table 3.4.1-1]** and in Table 12.8-2.

Abutment Height (Feet)	h_{eq} (Feet)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Table 12.8-3
Equivalent Height, h_{eq}, of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

WisDOT policy item:

The equivalent height of soil for vehicular loading on retaining walls parallel to traffic shall be 2.0 feet, regardless of the wall height. For standard unit weight of soil equal to 120 pcf, the resulting live load surcharge is 240 psf.

For abutments without reinforced concrete approaches, the equivalent height of soil for vehicular loading on abutments shall be based on Table 12.8-3. For abutments with reinforced concrete approaches, one half of the equivalent height of soil shall be used to calculate the horizontal load on the abutment.

12.8.4 Other Abutment Design Parameters

The equivalent fluid unit weights of soils are as presented in **LRFD [Table 3.11.5.5-1]**. Values are presented for loose sand or gravel, medium dense sand or gravel, and dense sand or gravel. Values are also presented for level or sloped backfill and for at-rest or active soil conditions.

Table 12.8-4 presents other parameters used in the design of abutments and wing walls. Standard details are based on the values presented in Table 12.8-4.

Description	Value
Bottom reinforcing steel cover	3.0 inches
Top reinforcing steel cover	2.0 inches
Unit weight of concrete	150 pcf
Concrete strength, f'_c	3.5 ksi
Reinforcing steel yield strength, f_y	60 ksi
Reinforcing steel modulus of elasticity, E_s	29,000 ksi
Unit weight of soil	120 pcf
Unit weight of structural backfill	120 pcf
Soil friction angle	30 degrees

Table 12.8-4
Other Parameters Used in Abutment Design

12.8.5 Abutment and Wing Wall Design in Wisconsin

The standard details for abutments and wing walls were developed as an envelope of the loading conditions produced by the standard superstructure types, span lengths and geometric conditions presented in this manual. Prior BOS approval is required and special consideration should be given to designs that are outside of the limits presented in the standard details. The loading conditions, material properties and design methods presented in this chapter should be used for these special designs.

WisDOT policy items:

The resistance of the wing pile to horizontal forces should not be included in the calculations for the wing capacity.

The passive earth resistance can only be developed if there is significant movement of the wing. The soil under the wing may settle or otherwise erode. Therefore, the resistance of the soil friction and the passive earth pressure should not be utilized in resisting the forces on wing walls.

In computing the weight of the approach slab, assume there is settlement under the approach slab and place one-half of the weight of the slab on the abutment. An unfactored dead load value of 1.2 klf shall be used for concrete approach slabs and 2.0 klf for structural approach slabs. An unfactored live load value of 0.900 klf shall be applied to abutment approach slabs when used. Approach reactions shall act along the centroid of the foundation.

The dynamic load allowance shall be applied to the live load for all abutment elements located above the ground line per **LRFD [3.6.2]**.

- Remove the material either completely or partially. This procedure is practical if the foundation depth is less than 15 feet and above the water table.
- Use lightweight embankment materials. Lightweight materials (fly ash, expanded shale and cinders) have been used with apparent success for abutment embankment construction to lessen the load on the foundation materials.

Abutment backfill practices that help minimize either settlement or swell include the following:

- Use of select materials
- Placement of relatively thin 4- to 6-inch layers
- Strict control of moisture and density
- Proper compaction
- Installation of moisture barriers

It is generally recognized by highway and bridge engineers that bridge abutments cause relatively few of the problems associated with bridge approaches. Proper drainage needs to be provided to prevent erosion of embankment or subgrade material that could cause settlement of the bridge approach. It is essential to provide for the removal of surface water that leaks into the area behind the abutment by using weepholes and/or drain tile. In addition, water infiltration between the approach slab and abutment body and wings must be prevented.

Reinforced concrete approach slabs are the most effective means for controlling surface irregularities caused by settlement. It is also important to allow enough expansion movement between the approach slab and the approach pavement to prevent horizontal thrust on the abutment.

The bridge designer should determine if a structural approach slab is required and coordinate details with the roadway engineer. Usage of structural approach slabs is currently based on road functional classifications and considerations to traffic volumes (AADT), design speeds, and settlement susceptibility.

WisDOT policy item:

Structural approach slabs shall be used on all bridges carrying traffic volumes greater than 3500 AADT in the future design year. Structural approach slabs are not required on buried structures and culverts and should not be used on rehabilitation projects. Other locations can be considered with the approval of the Chief Structural Design Engineer.

Standards for Structural Approach Slab for Type A1, A3, and A4 Abutments and Structural Approach Slab Details for Type A1, A3, and A4 Abutments are available for guidance.

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C_D = Lateral drag coefficient (dimensionless), as presented in Table 13.4-3

Angle Between the Flow Direction and the Pier's Longitudinal Axis	C_D
0°	0.0
5°	0.5
10°	0.7
20°	0.9
≥ 30°	1.0

Table 13.4-3
Lateral Drag Coefficient Values

The lateral drag force shall be computed as the product of lateral stream pressure and the projected exposed pier area. Use the water depth and velocity at flood stage with the force acting at one-half the water depth.

Normally the force of flowing water on piers does not govern the pier design.

13.4.7 Buoyancy

Buoyancy, a component of water load WA, is specified in **LRFD [3.7.2]** and is taken as the sum of the vertical components of buoyancy acting on all submerged components. The footings of piers in the floodplain are to be designed for uplift due to buoyancy.

Full hydrostatic pressure based on the water depth measured from the bottom of the footing is assumed to act on the bottom of the footing. The upward buoyant force equals the volume of concrete below the water surface times the unit weight of water. The effect of buoyancy on column design is usually ignored. Use high water elevation when analyzing the pier for over-turning. Use low water elevation to determine the maximum vertical load on the footing.

The submerged weight of the soil above the footing is used for calculating the vertical load on the footing. Typical values are presented in Table 13.4-4.

	Submerged Unit Weight, γ (pcf)				
	Sand	Sand & Gravel	Silty Clay	Clay	Silt
Minimum (Loose)	50	60	40	30	25
Maximum (Dense)	85	95	85	70	70

Table 13.4-4
Submerged Unit Weights of Various Soils

13.4.8 Ice

Forces from floating ice and expanding ice, IC, do not act on a pier at the same time. Consider each force separately when applying these design loads.

For all ice loads, investigate each site for existing conditions. If no data is available, use the following data as the minimum design criteria:

- Ice pressure = 32 ksf
- Minimum ice thickness = 12"
- Height on pier where force acts is at the 2-year high water elevation. If this value is not available, use the elevation located midway between the high and measured water elevations.
- Pier width is the projection of the pier perpendicular to stream flow.

Slender and flexible piers shall not be used in regions where ice forces are significant, unless approval is obtained from the WisDOT Bureau of Structures.

13.4.8.1 Force of Floating Ice and Drift

Ice forces on piers are caused by moving sheets or flows of ice striking the pier.

There is not an exact method for determining the floating ice force on a pier. The ice crushing strength primarily depends on the temperature and grain size of the ice. **LRFD [3.9.2.1]** sets the effective ice crushing strength at between 8 and 32 ksf.

The horizontal force caused by moving ice shall be taken as specified in **LRFD [3.9.2.2]**, as follows:

$$F = F_c = C_a ptw$$

$$C_a = \left(\frac{5t}{w} + 1\right)^{0.5}$$

Where:

p	=	Effective ice crushing strength (ksf)
t	=	Ice thickness (ft)
w	=	Pier width at level of ice action (ft)

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E13-1 Hammerhead Pier Design Example

This example shows design calculations conforming to the ***AASHTO LRFD Bridge Design Specifications (Seventh Edition - 2015 Interim)*** as supplemented by the *WisDOT Bridge Manual* The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-1.1 Obtain Design Criteria

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

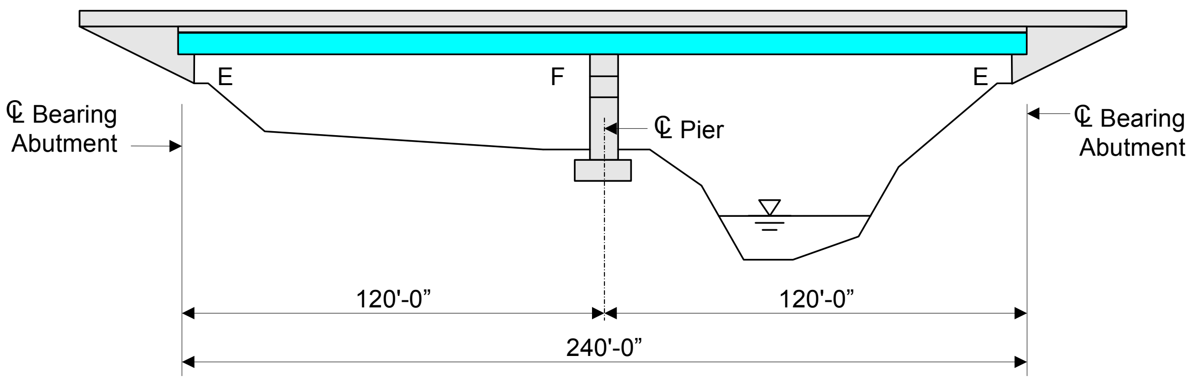


Figure E13-1.1-1
Bridge Elevation

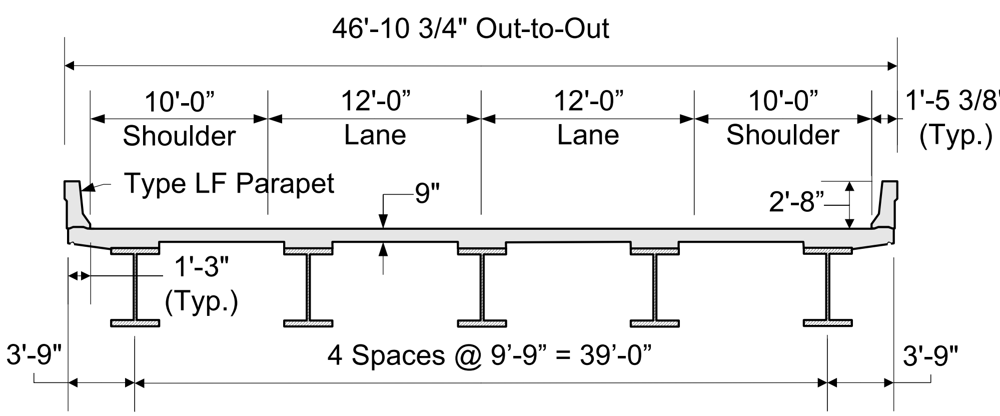


Figure E13-1.1-2
Bridge Cross Section

$As_{CD} := 9 \cdot As_{No11} + 9 \cdot As_{No10}$ $\boxed{As_{CD} = 25.45}$ in^2

Is $As_{CD} \geq Ast_{CD}$? $\boxed{check = "OK"}$

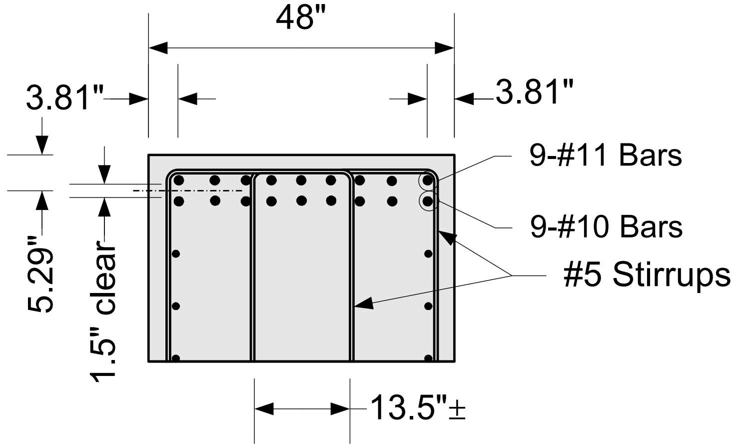


Figure E13-1.8-5
Cap Reinforcement at Tension Tie CD

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

For the top reinforcement past the first interior girder, the required area of tension tie reinforcement, A_{st}, in Tie DE for two lanes loaded is calculated as follows:

$\boxed{Pu_{DE\_1} = 800.79}$ kips

$\boxed{\phi = 0.9}$

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

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

$As_{DE} := 9 \cdot As_{No11} + 4 \cdot As_{No10}$ $\boxed{As_{DE} = 19.12}$ in^2

Is $As_{DE} \geq Ast_{DE}$? $\boxed{check = "OK"}$

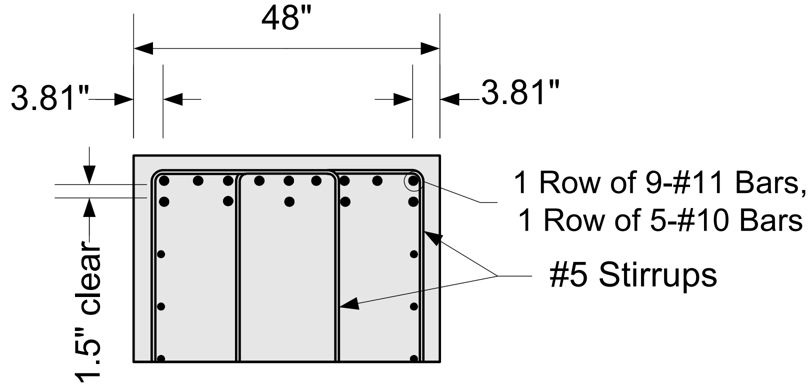


Figure E13-1.8-6
Cap Reinforcement at Tension Tie DE

E13-1.8.4 Calculate the Stirrup Reinforcement

The vertical tension ties DJ must resist a factored tension of force as shown below. The controlling force occurs with one lane loaded. This tension force will be resisted by stirrups with in the specified length of the pier cap. Note that any tension ties located directly over the column do not require stirrup design.

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

$$n := \frac{Pu}{\phi \cdot A_{st} \cdot f_y}$$

Try number 5 bars, with four legs.

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

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

$$n_{DJ} := \frac{Pu_{DJ\_1}}{\phi \cdot Ast \cdot f_y} \qquad n_{DJ} = 9.62$$

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

The length over which the stirrup shall be distributed is from the face of the column to half way between girders 4 and 5.

$\boxed{S = 9.75}$ feet

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

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

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

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

Crack control in disturbed regions:

$$\frac{A_{st}}{bs} \geq 0.003$$

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

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

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

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

$As_{DJ} := L_{DJ} \cdot Ast \cdot \frac{12}{8}$ $\boxed{As_{DJ} = 12.66}$ in^2

Therefore use No. 5 double-legged stirrups at 8 inch spacing in the pier cap.

E13-1.8.5 Compression Strut Capacity - Bottom Strut

After the tension tie reinforcement has been designed, the next step is to check the capacity of the compressive struts in the pier cap versus the limiting compressive stress. Strut IJ carries the highest bottom compressive force when one lane is loaded. Strut IJ is anchored by Node J, which also anchors ties DJ and strut EJ, From the geometry of the idealized internal truss, the smallest angle is between Tie DJ and Strut IJ:

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

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

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

Based on the design of the tension tie reinforcement, the tensile strain in Tie DJ is:

$$\varepsilon_s := \frac{P_u}{A_{st} \cdot E_s}$$

$E_s := 29000$ ksi

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

$\boxed{L_{DJ} = 6.88}$ feet

$\boxed{s_{stir} = 8}$ inches

$$Ast_{DJ} := \frac{L_{DJ} \cdot 12}{s_{stir}} \cdot Ast \qquad \boxed{Ast_{DJ} = 12.66} \quad in^2$$

$$\varepsilon_s := \frac{Pu_{DJ\_1}}{Ast_{DJ} \cdot E_s} \qquad \boxed{\varepsilon_s = 0.00174} \quad in/in$$

Therefore, the principal strain, ε_1, can be determined **LRFD [5.6.3.3.3]**:

$$\varepsilon_1 := \varepsilon_s + (\varepsilon_s + 0.002) \cdot \cot(\alpha_s)^2 \qquad \boxed{\varepsilon_1 = 0.00184} \quad in/in$$

The limiting compressive stress, f_{cu}, in the strut can also be computed **LRFD [5.6.3.3.3]**:

$$f_{cu} = \frac{f'_c}{0.8 + 170 \cdot \varepsilon_1} \le 0.85 \cdot f'_c$$

$$f_{cu1} := \frac{f'_c}{0.8 + 170 \cdot \varepsilon_1} \qquad \boxed{f_{cu1} = 3.15} \quad ksi$$

$$f_{cu2} := 0.85 \cdot f'_c \qquad \boxed{f_{cu2} = 2.98} \quad ksi$$

$$f_{cu} := \min(f_{cu1}, f_{cu2}) \qquad \boxed{f_{cu} = 2.98} \quad ksi$$

The nominal resistance of Strut IJ is computed based on the limiting stress, f_{cu}, and the strut dimensions. The centroid of the strut was assumed to be at $\boxed{centroid_{bot} = 4.5}$ inches vertically from the bottom face. Therefore, the thickness of the strut perpendicular to the sloping bottom face is:

$$t_{IJ} := 2 \cdot centroid_{bot} \cdot \cos(\theta) \qquad \boxed{t_{IJ} = 8.88} \quad inches$$

$$w_{IJ} := W_{cap} \cdot 12 \qquad \boxed{w_{IJ} = 48} \quad inches$$

$$Acs_{IJ} := t_{IJ} \cdot w_{IJ} \qquad \boxed{Acs_{IJ} = 426.27} \quad in^2$$

$$Pn_{IJ} := f_{cu} \cdot Acs_{IJ} \qquad \boxed{Pn_{IJ} = 1268.15} \quad kips$$

$\phi c_{STM} := 0.7$

$Pr_{IJ} := \phi c_{STM} \cdot Pn_{IJ}$ $\boxed{Pr_{IJ} = 887.71}$ kips

$\boxed{\left|Pu_{IJ\_1}\right| = 811.55}$ kips

Is $Pr_{IJ} \geq Pu_{IJ\_1}$? $\boxed{check = \text{"OK"}}$

E13-1.8.6 Compression Strut Capacity - Diagonal Strut

Strut DI carries the highest diagonal compressive force when two lanes are loaded. Strut DI is anchored by Node D, which also anchors ties CD, DE and DJ, From the geometry of the idealized internal truss, the smallest angle between Tie CD and Strut DI:

$\alpha_s := \operatorname{atan}\left(\frac{DI_v}{DI_h}\right)$ $\boxed{\alpha_s = 64.38 \cdot deg}$

$\theta := 90deg - \alpha_s$ $\boxed{\theta = 25.62 \cdot deg}$

$\boxed{Pu_{DI\_2} = -1471.41}$ kips

The tensile strain in Ties CD and DE are calculated as follows. The average of these two strains is used to check the capacity of Strut DI.

$\boxed{Pu_{CD\_2} = 1371.6}$ kips $\boxed{As_{CD} = 25.45}$ in^2

$\boxed{Pu_{DE\_2} = 735.42}$ kips $\boxed{As_{DE} = 19.12}$ in^2

$\varepsilon_{sCD\_2} := \frac{Pu_{CD\_2}}{As_{CD} \cdot E_s}$ $\boxed{\varepsilon_{sCD\_2} = 0.00186}$ $\frac{in}{in}$

$\varepsilon_{sDE\_2} := \frac{Pu_{DE\_2}}{As_{DE} \cdot E_s}$ $\boxed{\varepsilon_{sDE\_2} = 0.00133}$ $\frac{in}{in}$

$\varepsilon_{s\_ave} := \frac{\varepsilon_{sCD\_2} + \varepsilon_{sDE\_2}}{2}$ $\boxed{\varepsilon_{s\_ave} = 0.00159}$ $\frac{in}{in}$

Therefore, the principal strain, ε_1, can be determined **LRFD [5.6.3.3.3]**:

$$\varepsilon_1 := \varepsilon_{s\_ave} + (\varepsilon_{s\_ave} + 0.002)\cdot\cot(\alpha_s)^2 \qquad \boxed{\varepsilon_1 = 0.00242} \qquad \frac{in}{in}$$

The limiting compressive stress, f_{cu}, in the strut can also be computed **LRFD [5.6.3.3.3]**:

$$f_{cu} = \frac{f'_c}{0.8 + 170\cdot\varepsilon_1} \le 0.85\cdot f'_c$$

$$f_{cu1} := \frac{f'_c}{0.8 + 170\cdot\varepsilon_1} \qquad \boxed{f_{cu1} = 2.89} \qquad ksi$$

$$f_{cu2} := 0.85\cdot f'_c \qquad \boxed{f_{cu2} = 2.98} \qquad ksi$$

$$f_{cu} := \min(f_{cu1}, f_{cu2}) \qquad \boxed{f_{cu} = 2.89} \qquad ksi$$

The cross sectional dimension of Strut DI in the plane of the pier is calculated as follows. Note that for skewed bearings, the length of the bearing is the projected length along the centerline of the pier cap.

$\boxed{L_{brng} = 26}$ inches

$\boxed{W_{brng} = 18}$ inches

$\boxed{centroid_{top} = 5.5}$ inches

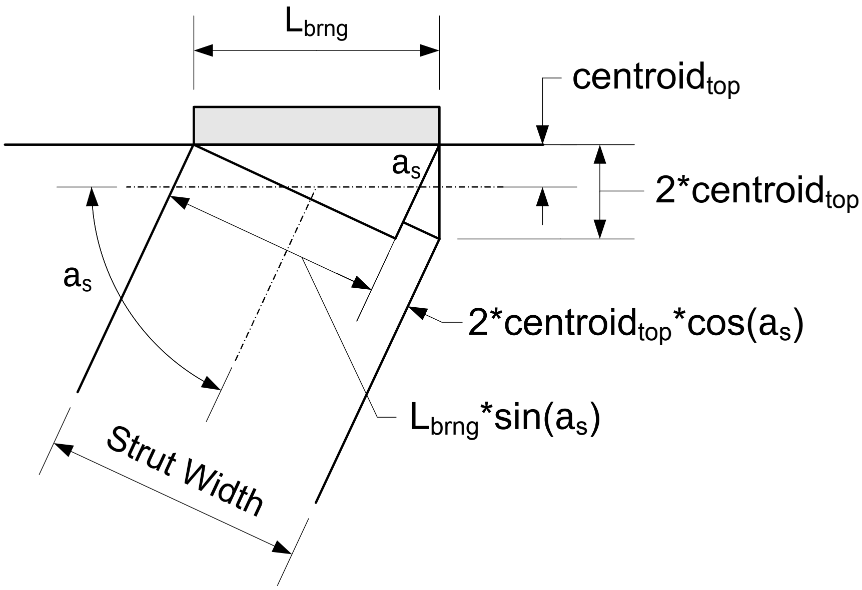


Figure E13-1.8-7
Compression Strut Width

$t_{DI} := L_{brng} \cdot \sin(\alpha_s) + 2 \cdot centroid_{top} \cdot \cos(\alpha_s)$ $\boxed{t_{DI} = 28.2}$ in

The effective compression strut width around each stirrup is:

$d_{bar11} := 1.410$ inches

$w_{ef} := 2 \cdot 6 \cdot d_{bar11}$ $\boxed{w_{ef} = 16.92}$ in

The effective spacing between the 4 legs of the stirrups is 13.5 inches, which is less than the value calculated above. Therefore, the entire cap width can be used for the effective strut width.

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

The nominal resistance of Strut DI is computed based on the limiting stress, f_{cu}, and the strut dimensions.

$Acs_{DI} := t_{DI} \cdot w_{DI}$ $\boxed{Acs_{DI} = 1353.61}$ in^2

$Pn_{DI} := f_{cu} \cdot Acs_{DI}$ $\boxed{Pn_{DI} = 3911.99}$ kips

$\boxed{\phi c_{STM} = 0.7}$

$Pr_{DI} := \phi c_{STM} \cdot Pn_{DI}$ $\boxed{Pr_{DI} = 2738.4}$ kips

$\boxed{\left|Pu_{DI\_2}\right| = 1471.41}$ kips

Is $Pr_{DI} \geq \left|Pu_{DI\_2}\right|$? $\boxed{check = \text{"OK"}}$

E13-1.8.7 Check the Anchorage of the Tension Ties

12 No. 11 longitudinal bars along the top of the pier cap must be developed at the inner edge of the bearing at Node E (the edge furthest from the end of the member). Based on **Figure E13-1.8-8**, the embedment length that is available to develop the bar beyond the edge of the bearing is:

L_{devel} = (distance from end to Node) + (bearing block width/2) - (cover)

$\boxed{L_{cap} = 46.5}$ feet

$\boxed{S = 9.75}$ feet

$\boxed{L_{brng} = 26}$ inches

$\boxed{Cover_{cp} = 2.5}$ inches

$$L_{devel} := \frac{L_{cap} - S\cdot(ng-1)}{2}\cdot 12 + \frac{L_{brng}}{2} - Cover_{cp} \qquad \boxed{L_{devel} = 55.5} \quad \text{in}$$

The basic development length for straight No. 11 and No. 10 bars with spacing less than 6", As(provided)/As(required) < 2, uncoated top bar, per **Wis Bridge Manual Table 9.9-1** is:

$\boxed{L_{d11} := 9.5}$ ft $\qquad \boxed{L_{d11}\cdot 12 = 114}$ in

$\boxed{L_{d10} := 7.75}$ ft $\qquad \boxed{L_{d10}\cdot 12 = 93}$ in

Therefore there is not sufficient development length for straight bars. Check the hook development length. The base hook development length for 90º hooked No.11 and #10 bars per **LRFD [5.11.2.4]** is:

$$L_{hb11} := \frac{38.0\cdot d_{bar11}}{\sqrt{f'_c}} \qquad \boxed{L_{hb11} = 28.64} \quad \text{in}$$

$$L_{hb10} := \frac{38.0\cdot d_{bar10}}{\sqrt{f'_c}} \qquad \boxed{L_{hb10} = 25.8} \quad \text{in}$$

The length available is greater than the base hook development length, therefore the reduction factors do not need to be considered. Hook both the top 9 bars and the bottom layer 5 bars. The remaining 4 bottom layer bars can be terminated 7.75 feet from the inside edge of the bearings at girders 2 and 4.

In addition, the tension ties must be spread out sufficiently in the effective anchorage area. The centroid of the tension ties is $\boxed{centroid_{top} = 5.5}$ inches below the top of the pier cap. Therefore, the effective depth of the anchorage area is 11 inches. The nodal zone stress to anchor the tension tie is:

$\boxed{Pu_{DE\_1} = 800.79}$ kips

$\boxed{centroid_{top} = 5.5}$ inches

$$f_c := \frac{Pu_{DE\_1}}{2\cdot centroid_{top}\cdot W_{cap}\cdot 12} \qquad \boxed{f_c = 1.52} \quad \text{ksi}$$

This nodal region anchors a one direction tension tie, and Node E is classified as a CCT node. The limiting nodal zone stress presented in **Table 13-1.8-1** is:

$\boxed{0.75\cdot\phi\cdot f'_c = 2.36}$ ksi

Is $0.75\cdot\phi\cdot f'_c \ge f_c$? $\qquad \boxed{check = \text{"OK"}}$

Therefore, the requirement for the nodal zone stress limit in the anchorage area is satisfied.

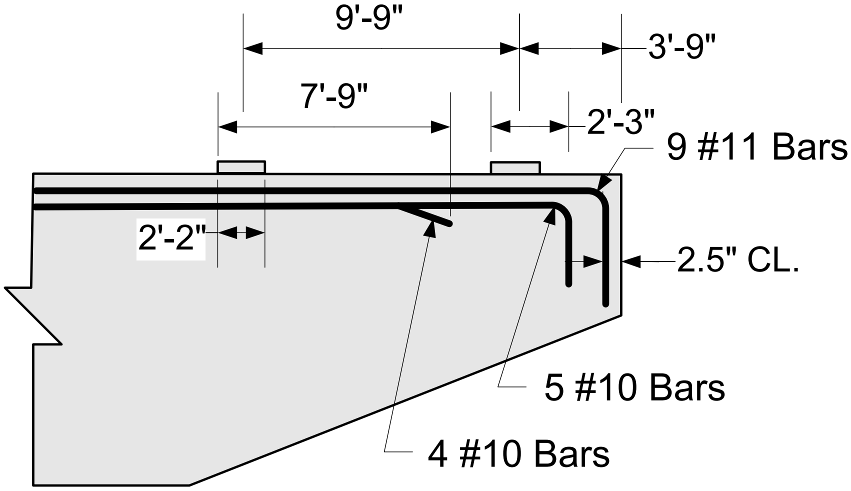


Figure E13-1.8-8
Anchorage of Tension Tie

E13-1.8.8 Provide Crack Control Reinforcement

In the disturbed regions, the minimum ratio of reinforcement to the gross concrete area is 0.003 in each direction, and the spacing of the bars in these grids must not exceed 12 inches, **LRFD [5.6.3.6]**. Therefore the required crack control reinforcement within a 1 foot section is:

$As_{crack} := 0.003 \cdot (12) \cdot W_{cap} \cdot 12$ $\boxed{As_{crack} = 1.73}$ in^2

Use 4 - No. 7 horizontal bars at 12 inch spacing in the vertical direction

$As_{No7} := 0.6013$ $\boxed{4 \cdot As_{No7} = 2.41}$ in^2

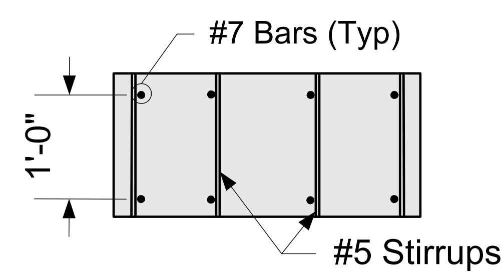


Figure E13-1.8-9
Crack Control Reinforcement - Option 1

OR If we assume 6-inch vertical spacing

$As_{crack} := 0.003 \cdot (6) \cdot W_{cap} \cdot 12$ $\boxed{As_{crack} = 0.86}$ in^2

$2 \cdot As_{No7} = 1.2$ in^2

Is $2 \cdot As_{No7} \geq As_{crack}$? $check = \text{"OK"}$

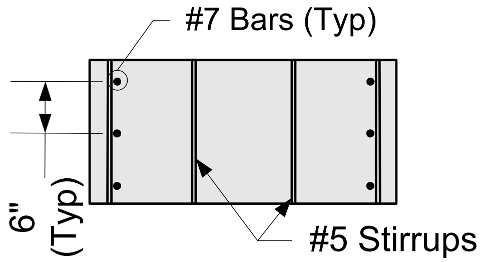


Figure E13-1.8-10
Crack Control Reinforcement - Option 2

This 6-inch spacing for the number 7 temperature and shrinkage reinforcement is also used along the bottom of the cap.

The stirrups are spaced at, $s_{stir} = 8$ inches. Therefore the required crack control reinforcement within this spacing is:

$$As_{crack2} := 0.003 \cdot (s_{stir}) \cdot W_{cap} \cdot 12 \qquad As_{crack2} = 1.15 \quad in^2$$

4 legs of No.5 stirrups at $s_{stir} = 8$ inch spacing in the horizontal direction

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

Is $4 \cdot As_{No5} \geq As_{crack2}$? $check = \text{"OK"}$

E13-1.8.9 Summary of Cap Reinforcement

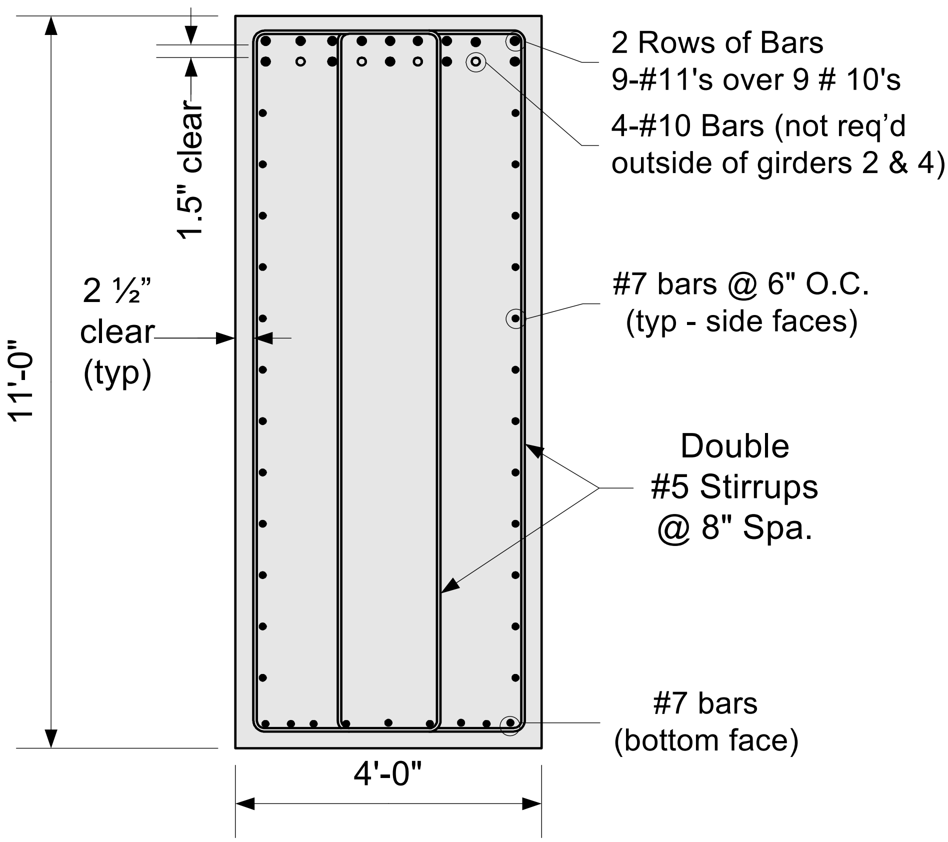


Figure E13-1.8-11
Pier Cap Design Summary

E13-1.9 Design Pier Column

As stated in E13-1.7, the critical section in the pier column is where the column meets the footing, or at the column base. The governing force effects and their corresponding limit states were determined to be:

Strength V

$\boxed{Ax_{colStrV} = 2054.87}$ kips

$\boxed{MuT_{colStrV} = 8789.59}$ kip-ft

$\boxed{MuL_{colStrV} = 2333.6}$ kip-ft

Strength III

$\boxed{VuT_{col} = 76.45}$ kips

Strength V

$\boxed{VuL_{col} = 105.37}$ kips

A preliminary estimate of the required section size and reinforcement is shown in Figure E13-1.9-1.

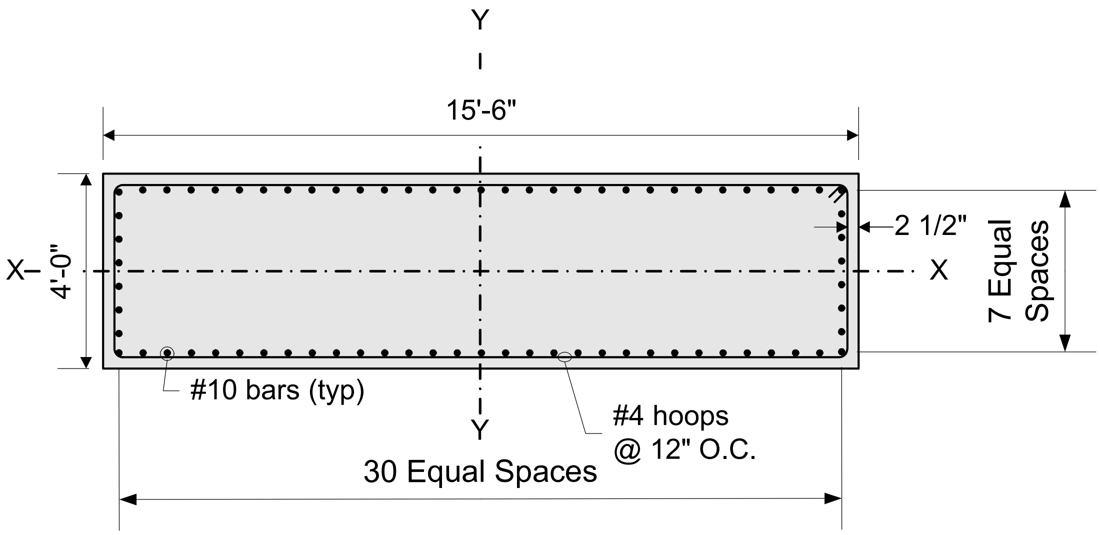


Figure E13-1.9-1
Preliminary Pier Column Design

E13-1.9.1 Design for Axial Load and Biaxial Bending (Strength V):

The preliminary column reinforcing is show in Figure E13-1.9-1 and corresponds to #10 bars equally spaced around the column perimeter. **LRFD [5.7.4.2]** prescribes limits (both maximum and minimum) on the amount of reinforcing steel in a column. These checks are performed on the preliminary column as follows:

$Num\_bars := 74$ $bar\_area10 := 1.27$ 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

$\frac{A_{s\_col}}{A_{g\_col}} = 0.0105$ $0.0105 \le 0.08$ (max. reinf. check) OK

$\frac{0.135\cdot f'_c}{f_y} = 0.008$ (but need not be greater than 0.015) $0.0105 \ge 0.008$ (min. reinf. check) OK

The column slenderness ratio (Kl_u/r) about each axis of the column is computed below in order to assess slenderness effects. Note that the Specifications only permit the following approximate evaluation of slenderness effects when the slenderness ratio is below 100.

For this pier, the unbraced lengths (l_{ux}, l_{uy}) used in computing the slenderness ratio about each axis is the full pier height. This is the height from the top of the footing to the top of the pier cap (26 feet). The effective length factor in the longitudinal direction, K_x, is taken equal to 2.1. This assumes that the superstructure has no effect on restraining the pier from buckling. In essence, the pier is considered a free-standing cantilever in the longitudinal direction. The effective length factor in the transverse direction, K_y, is taken to equal 1.0.

The radius of gyration (r) about each axis can then be computed as follows:

$$I_{xx} := \frac{(L_{col} \cdot 12) \cdot (W_{col} \cdot 12)^3}{12} \qquad \boxed{I_{xx} = 1714176} \qquad in^4$$

$$I_{yy} := \frac{(W_{col} \cdot 12) \cdot (L_{col} \cdot 12)^3}{12} \qquad \boxed{I_{yy} = 25739424} \qquad in^4$$

$$r_{xx} := \sqrt{\frac{I_{xx}}{A_{g\_col}}} \qquad \boxed{r_{xx} = 13.86} \qquad in$$

$$r_{yy} := \sqrt{\frac{I_{yy}}{A_{g\_col}}} \qquad \boxed{r_{yy} = 53.69} \qquad in$$

The slenderness ratio for each axis now follows:

$$K_x := 2.1$$

$$K_y := 1.0$$

$$L_u := (H_{col} + H_{cap}) \cdot 12 \qquad \boxed{L_u = 312} \qquad in$$

$$\boxed{\frac{K_x \cdot L_u}{r_{xx}} = 47.28} \qquad 47.28 < 100 \qquad \text{OK}$$

$$\boxed{\frac{K_y \cdot L_u}{r_{yy}} = 5.81} \qquad 5.81 < 100 \qquad \text{OK}$$

LRFD [5.7.4.3] permits the slenderness effects to be ignored when the slenderness ratio is less than 22 for members not braced against side sway. It is assumed in this example that the pier is not braced against side sway in either its longitudinal or transverse directions. Therefore, slenderness will be considered for the pier longitudinal direction only (i.e., about the "X-X" axis).

In computing the amplification factor that is applied to the longitudinal moment, which is the enc result of the slenderness effect, the column stiffness (EI) about the "X-X" axis must be defined.

In doing so, the ratio of the maximum factored moment due to permanent load to the maximum factored moment due to total load must be identified (β_d).

From Design Step E13-1.7, it can be seen that the force effects contributing to the longitudinal moment are the live load braking force, the temperature force and wind on the structure and live load. None of these are permanent or long-term loads. Therefore, β_d is taken equal to zero for this design.

$$\beta_d := 0$$

$$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$$ **LRFD [C5.4.2.7]** $\boxed{E_c = 3587}$ ksi

$\boxed{E_s = 29000.00}$ ksi

$\boxed{I_{xx} = 1714176}$ in^4

I_s = Moment of Inertia of longitudinal steel about the centroidal axis (in^4)

$$I_s := \frac{\pi \cdot bar\_dia10^4}{64} \cdot (Num\_bars) + 2 \cdot 31 \cdot (bar\_area10) \cdot 20.37^2 \ ...$$
$$+ 4 \cdot (bar\_area10) \cdot 14.55^2 + 4 \cdot (bar\_area10) \cdot 8.73^2 + 4 \cdot (bar\_area10) \cdot 2.91^2$$

$\boxed{I_s = 34187}$ in^4

The column stiffness is taken as the greater of the following two calculations:

$$EI_1 := \frac{\frac{E_c \cdot I_{xx}}{5} + E_s \cdot I_s}{1 + \beta_d}$$ $\boxed{EI_1 = 2.22 \times 10^9}$ $k\text{-}in^2$

$$EI_2 := \frac{\frac{E_c \cdot I_{xx}}{2.5}}{1 + \beta_d}$$ $\boxed{EI_2 = 2.46 \times 10^9}$ $k\text{-}in^2$

$$EI := \max(EI_1, EI_2)$$ $\boxed{EI = 2.46 \times 10^9}$ $k\text{-}in^2$

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 [5.5.4.2.1]** permits the value of phi to be increased linearly to the value for flexure (0.90) as the section changes from compression controlled to tension controlled as defined in **LRFD [5.7.2.1]**. However, certain equations in the Specification still require the use of the phi factor for axial compression (0.75) even when the increase just described is permitted. Therefore, for the sake of clarity in this example, if phi may be increased it will be labeled separately from ϕ_{axial} identified above.

$As_{col} := 2.53$ in^2 per foot, based on #10 bars at 6-inch spacing

$b := 12$ inches $\alpha_1 := 0.85$ (for f'_c < 10.0 ksi) **LRFD [5.7.2.2]**

$a := \dfrac{As_{col} \cdot f_y}{\alpha_1 \cdot f'_c \cdot b}$ $\boxed{a = 4.25}$ 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

$\varepsilon_t := 0.005$ Lower strain limit for tension controlled sections, for f_y = 60 ksi

LRFD [**Table C5.7.2-1**]

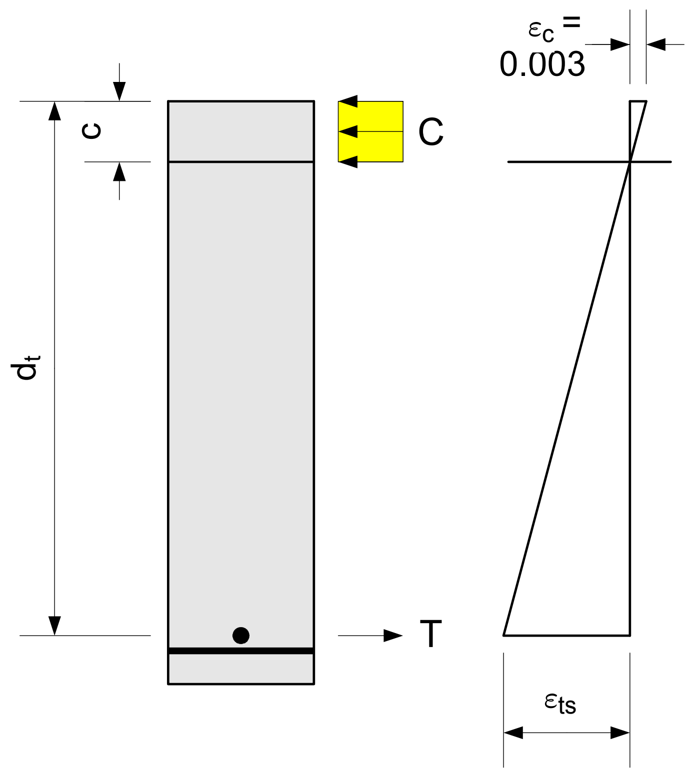


Figure E13-1.9-2

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:

$$P_e := \frac{\pi^2 \cdot EI}{(K_x \cdot L_u)^2} \qquad \boxed{P_e = 56539.53} \quad \text{kips}$$

$$\delta_s := \frac{1}{1 - \left(\frac{Ax_{colStrV}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_s = 1.04}$$

The final design forces at the base of the column for the Strength I limit state will be redefined as follows:

$$P_{u\_col} := Ax_{colStrV} \qquad \boxed{P_{u\_col} = 2054.87} \quad \text{kips}$$

$$M_{ux} := MuL_{colStrV} \cdot \delta_s \qquad \boxed{M_{ux} = 2431.8} \quad \text{kip-ft}$$

$$M_{uy} := MuT_{colStrV} \qquad \boxed{M_{uy} = 8789.59} \quad \text{kip-ft}$$

The assessment of the resistance of a compression member with biaxial flexure for strength limit states is dependent upon the magnitude of the factored axial load. This value determines which of two equations provided by the Specification are used.

If the factored axial load is less than ten percent of the gross concrete strength multiplied by the phi-factor for compression members (ϕ_{axial}), then the Specifications require that a linear interaction equation for only the moments is satisfied (**LRFD [Equation 5.7.4.5-3]**). Otherwise, an axial load resistance (P_{rxy}) is computed based on the reciprocal load method (**LRFD [Equation 5.7.4.5-1]**). In this method, axial resistances of the column are computed (using f_{Low_axial} if applicable) with each moment acting separately (i.e., P_{rx} with M_{ux}, P_{ry} with M_{uy}). These are used along with the theoretical maximum possible axial resistance (P_o multiplied by ϕ_{axial}) to obtain the factored axial resistance of the biaxially loaded column.

Regardless of which of the two equations mentioned in the above paragraph controls, commercially available software is generally used to obtain the moment and axial load resistances.

For this pier design, the procedure as discussed above is carried out as follows:

$$\boxed{0.10 \cdot \phi_{axial} \cdot f'_c \cdot A_{g\_col} = 2343.6} \quad \text{kips}$$

$$\boxed{P_{u\_col} = 2054.87} \quad \text{kips} \qquad P_{u\_col} < 2343.6K$$

Therefore, **LRFD [Equation 5.7.4.5-3]** will be used.

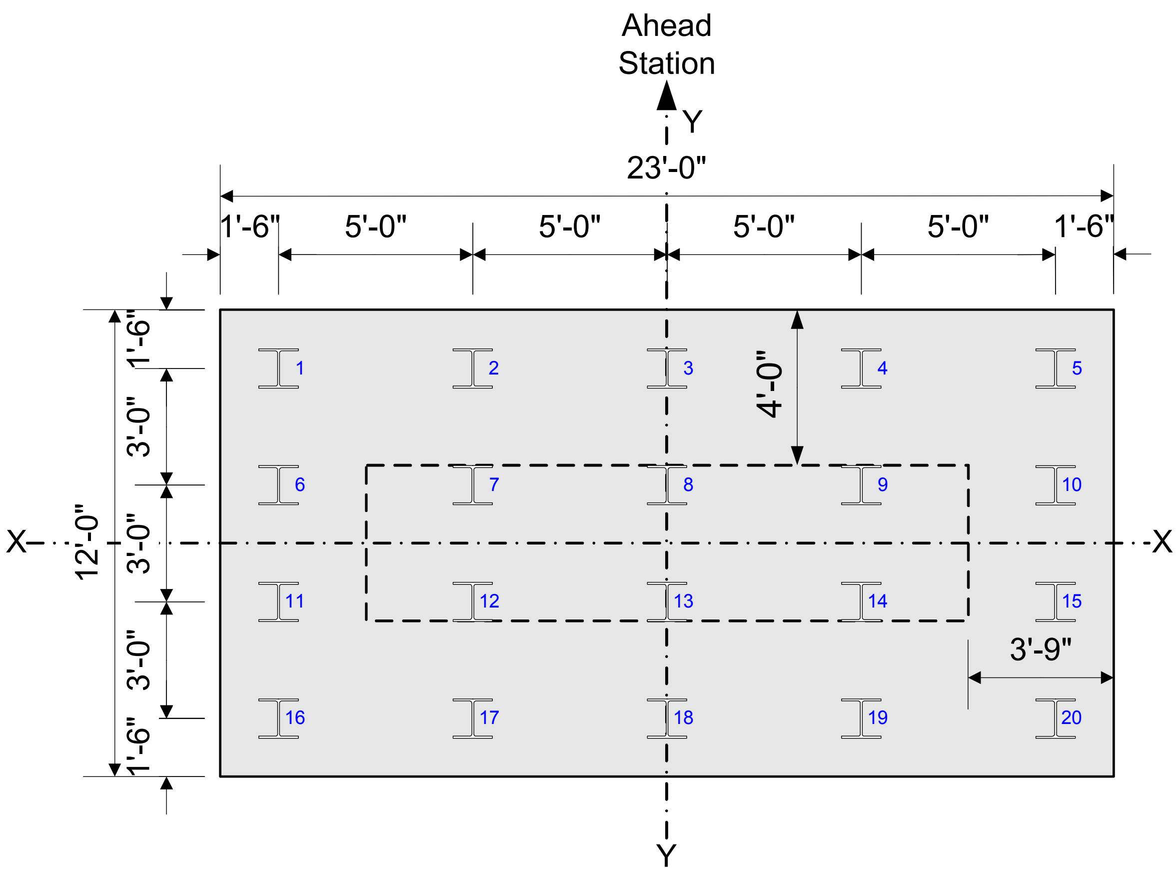


Figure E13-1.10-1

Pier Pile Layout

$N_p := 20$ Number of piles

$$S_{xx} := \frac{10 \cdot 4.5^2 + 10 \cdot 1.5^2}{4.5} \qquad \boxed{S_{xx} = 50} \quad ft^3$$

$$S_{yy} := \frac{8 \cdot 10^2 + 8 \cdot 5^2}{10} \qquad \boxed{S_{yy} = 100} \quad ft^3$$

Maximum pile reaction:

$\boxed{\phi_t = 0.9}$

$\boxed{P_e = 56539.53}$ kips (from column design)

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

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

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

$$\delta_{pile\_StrI} := \frac{1}{1 - \left(\frac{Pu2_{pile\_Str1}}{\phi_t \cdot P_e}\right)}$$

$\boxed{\delta_{pile\_StrI} = 1.07}$

$$Pu_p := \frac{Pu2_{pile\_Str1}}{N_p} + \frac{MuT2_{pile\_Str1}}{S_{yy}} + \frac{MuL2_{pile\_Str1} \cdot \delta_{pile\_StrI}}{S_{xx}}$$

$\boxed{Pu_p = 276.93}$ kips

$$Pu_{p\_tons} := \frac{Pu_p}{2}$$

$\boxed{Pu_{p\_tons} = 138.46}$ tons

From Wis Bridge Manual, Section 11.3.1.17.6, the vertical pile resistance of HP12x53 pile is :

$\boxed{Pr_{12x53} = 110}$ tons $\boxed{check = \text{"No Good"}}$

$\boxed{Pr_{12x53\_PDA} = 143}$ tons $\boxed{check = \text{"OK"}}$

Note: PDA with CAPWAP is typically used when it is more economical than modified Gates. This example uses PDA with CAPWAP only to illustrate that vertical pile reactions are satisfied and to minimize example changes due to revised pile values. The original example problem was based on higher pile values than the current values shown in Chapter 11,Table 11.3-5.

Minimum pile reaction (Strength V):

$\boxed{Pu_{pile\_StrV} = 2134.91}$ kips

$\boxed{MuT_{pile\_StrV} = 7670.61}$ kip-ft

$\boxed{MuL_{pile\_StrV} = 2333.6}$ kip-ft

$$\delta_{pile\_StrV} := \frac{1}{1 - \left(\frac{Pu_{pile\_StrV}}{\phi_t \cdot P_e}\right)}$$

$\boxed{\delta_{pile\_StrV} = 1.04}$

$$Pu_{min\_p} := \frac{Pu_{pile\_StrV}}{N_p} - \frac{MuT_{pile\_StrV}}{S_{yy}} - \frac{MuL_{pile\_StrV} \cdot \delta_{pile\_StrV}}{S_{xx}}$$

$\boxed{Pu_{min\_p} = -18.68}$ kips

Capacity for pile uplift is site dependant. Consult with the geotechnical engineer for allowable values.

The horizontal pile resistance of HP12x53 pile from the soils report is :

$Hr_{12x53} := 14$ kips/pile

Pile dimensions in the transverse (xx) and longitudinal (yy) directions:

$B_{xx} := 12.05$ inches

$B_{yy} := 11.78$ inches

Pile spacing in the transverse and longitudinal directions:

$Spa_{xx} := 5.0$ feet $\boxed{\dfrac{Spa_{xx}}{\dfrac{B_{xx}}{12}} = 4.98}$ Say: 5B

$Spa_{yy} := 3.0$ feet $\boxed{\dfrac{Spa_{yy}}{\dfrac{B_{yy}}{12}} = 3.06}$ Say: 3B

Use the pile multipliers from **LRFD [T-10.7.2.4-1]** to calculate the group resistance of the piles in each direction.

$Hr_{xx} := Hr_{12x53} \cdot 4 \cdot (1.0 + 0.85 + 0.70 \cdot 3)$

$\boxed{Hr_{xx} = 221.2}$ kips

$\boxed{HuT_{pileStrIII} = 76.45}$ kips

$Hr_{xx} \geq HuT_{pileStrIII}$

$\boxed{check = \text{"OK"}}$

$Hr_{yy} := Hr_{12x53} \cdot 5 \cdot (0.7 + 0.5 + 0.35 \cdot 2)$

$\boxed{Hr_{yy} = 133}$ kips

$\boxed{HuL_{pileStrV} = 105.37}$ kips

$Hr_{yy} \geq HuL_{pileStrV}$

$\boxed{check = \text{"OK"}}$

E13-1.11 - Design Pier Footing

In E13-1.7, the Strength I limit states was identified as the governing limit state for the design of the pier footing.

Listed below are the Strength I footing loads for one, two and three lanes loaded:

$\boxed{Pu1_{ftgStr1} = 2643.74}$ kips $\boxed{Pu2_{ftgStr1} = 2928.7}$ kips

$\boxed{MuT1_{ftgStr1} = 7267.81}$ kip-ft $\boxed{MuT2_{ftgStr1} = 7836.85}$ kip-ft

$\boxed{MuL1_{ftgStr1} = 1187.7}$ kip-ft $\boxed{MuL2_{ftgStr1} = 1856.29}$ kip-ft

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

$\boxed{MuT3_{ftgStr1} = 4541.55}$ kip-ft

$\boxed{MuL3_{ftgStr1} = 2315.94}$ kip-ft

The longitudinal moment given above must be magnified to account for slenderness of the column (see E13-1.9). The computed magnification factor and final factored forces are:

$$\delta_{s1\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu1_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s1\_ftgStr1} = 1.05}$$

$$\delta_{s2\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu2_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s2\_ftgStr1} = 1.06}$$

$$\delta_{s3\_ftgStr1} := \frac{1}{1 - \left(\frac{Pu3_{ftgStr1}}{\phi_t \cdot P_e}\right)} \qquad \boxed{\delta_{s3\_ftgStr1} = 1.07}$$

$$MuL1_{ftgStr1\delta} := \delta_{s1\_ftgStr1} \cdot MuL1_{ftgStr1} \qquad \boxed{MuL1_{ftgStr1\delta} = 1252.79} \text{ kip-ft}$$

$$MuL2_{ftgStr1\delta} := \delta_{s2\_ftgStr1} \cdot MuL2_{ftgStr1} \qquad \boxed{MuL2_{ftgStr1\delta} = 1969.65} \text{ kip-ft}$$

$$MuL3_{ftgStr1\delta} := \delta_{s3\_ftgStr1} \cdot MuL3_{ftgStr1} \qquad \boxed{MuL3_{ftgStr1\delta} = 2467.46} \text{ kip-ft}$$

$Mu_{ftg} := \max(Mu_{xx}, Mu_{yy})$ $\boxed{Mu_{ftg} = 198.15}$ 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:

$\phi_f := 0.90$

$\boxed{b = 12}$ in

$\boxed{f'_c = 3.5}$ ksi

$Rn := \dfrac{M_{Design} \cdot 12}{\phi_f \cdot b \cdot d_e^{\,2}}$ $\boxed{Rn = 0.175}$

$\rho := 0.85 \left(\dfrac{f'_c}{f_y}\right) \cdot \left(1.0 - \sqrt{1.0 - \dfrac{2 \cdot Rn}{0.85 \cdot f'_c}}\right)$ $\boxed{\rho = 0.00300}$

$As_{ftg} := \rho \cdot b \cdot d_e$ $\boxed{As_{ftg} = 1.28}$ in^2 per foot

Required bar spacing = $\boxed{\dfrac{bar\_area8}{As_{ftg}} \cdot 12 = 7.41}$ in

Use #8 bars @ $bar\_space := 7$

$A_{sftg} := bar\_area8 \cdot \left(\dfrac{12}{bar\_space}\right)$ $\boxed{A_{sftg} = 1.35}$ in^2 per foot

Is $A_{sftg} \geq As_{ftg}$? $\boxed{check = \text{"OK"}}$

Similar calculations can be performed for the reinforcing in the longitudinal direction. The effective depth for this reinforcing is calculated based on the longitudinal bars resting directly on top of the transverse bars.

E13-1.11.2 Punching Shear Check

The factored force effects from E13-1.7 for the punching shear check at the column are:

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

$\boxed{MuT3_{ftgStr1} = 4541.55}$ kip-ft

$\boxed{MuL3_{ftgStr1\delta} = 2467.46}$ kip-ft

$$Pu3 = \begin{pmatrix} 251 & 228.29 & 205.58 & 182.87 & 160.17 \\ 218.1 & 195.39 & 172.68 & 149.97 & 127.27 \\ 185.2 & 162.49 & 139.78 & 117.08 & 94.37 \\ 152.3 & 129.59 & 106.88 & 84.18 & 61.47 \end{pmatrix}$$

$\boxed{Pu3_{pile} = 251}$ kips

With the applied factored loads determined, the next step in the column punching shear check is to define the critical perimeter, b_o. The Specifications require that this perimeter be minimized, but need not be closer than $d_v/2$ to the perimeter of the concentrated load area. In this case, the concentrated load area is the area of the column on the footing as seen in plan.

The effective shear depth, d_v, must be defined in order to determine b_o and the punching (or two-way) shear resistance. An average effective shear depth should be used since the two-way shear area includes both the "X-X" and "Y-Y" sides of the footing. In other words, d_{ex} is not equal to d_{ey}, therefore d_{vx} will not be equal to d_{vy}. This is illustrated as follows assuming a 3'-6" footing with #8 reinforcing bars at 6" on center in both directions in the bottom of the footing:

$\boxed{b = 12}$ in

$h_{ftg} := H_{ftg} \cdot 12$ $\boxed{h_{ftg} = 42}$ in

$A_{s\_ftg} := 2 \cdot (bar\_area8)$ $\boxed{A_{s\_ftg} = 1.58}$ in^2 per foot width

Effective depth for each axis:

$\boxed{Cover_{fb} = 6}$

$d_{ey} := h_{ftg} - Cover_{fb} - \frac{bar\_diam8}{2}$ $\boxed{d_{ey} = 35.5}$ in

$d_{ex} := h_{ftg} - Cover_{fb} - bar\_diam8 - \frac{bar\_diam8}{2}$ $\boxed{d_{ex} = 34.5}$ in

Effective shear depth for each axis:

$T_{ftg} := A_{s\_ftg} \cdot f_y$ $\boxed{T_{ftg} = 94.8}$ kips

$a_{ftg} := \dfrac{T_{ftg}}{\alpha_1 \cdot f'_c \cdot b}$ $\boxed{a_{ftg} = 2.66}$ in

$d_{vx} := \max\left(d_{ex} - \dfrac{a_{ftg}}{2}, 0.9 \cdot d_{ex}, 0.72 \cdot h_{ftg}\right)$ $\boxed{d_{vx} = 33.17}$ in

$d_{vy} := \max\left(d_{ey} - \dfrac{a_{ftg}}{2}, 0.9 \cdot d_{ey}, 0.72 \cdot h_{ftg}\right)$ $\boxed{d_{vy} = 34.17}$ in

Average effective shear depth:

$d_{v\_avg} := \dfrac{d_{vx} + d_{vy}}{2}$ $\boxed{d_{v\_avg} = 33.67}$ in

With the average effective shear depth determined, the critical perimeter can be calculated as follows:

$b_{col} := L_{col} \cdot 12$ $\boxed{b_{col} = 186}$ in

$t_{col} := W_{col} \cdot 12$ $\boxed{t_{col} = 48}$ in

$b_o := 2\left[b_{col} + 2 \cdot \left(\dfrac{d_{v\_avg}}{2}\right)\right] + 2 \cdot \left[t_{col} + 2 \cdot \left(\dfrac{d_{v\_avg}}{2}\right)\right]$ $\boxed{b_o = 602.69}$ in

The factored shear resistance to punching shear is the smaller of the following two computed values:

$\beta_c := \dfrac{b_{col}}{t_{col}}$ $\boxed{\beta_c = 3.88}$

$V_{n\_punch1} := \left(0.063 + \dfrac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_{v\_avg})$ $\boxed{V_{n\_punch1} = 3626.41}$ kips

$V_{n\_punch2} := 0.126 \cdot \left(\sqrt{f'_c}\right) \cdot (b_o) \cdot (d_{v\_avg})$ $\boxed{V_{n\_punch2} = 4783.77}$ kips

$V_{n\_punch} := \min\left(V_{n\_punch1}, V_{n\_punch2}\right)$ $\boxed{V_{n\_punch} = 3626.41}$ kips

$\phi_v = 0.9$

$V_{r\_punch} := \phi_v \cdot \left(V_{n\_punch}\right)$ $\boxed{V_{r\_punch} = 3263.77}$ kips

With the factored shear resistance determined, the applied factored punching shear load will be computed. This value is obtained by summing the loads in the piles that are outside of the critical perimeter. As can be seen in Figure E13-1.11-2, this includes Piles 1 through 5, 6, 10,11, 15, and 16 through 20. These piles are entirely outside of the critical perimeter. If part of a pile is inside the critical perimeter, then only the portion of the pile load outside the critical perimeter is used for the punching shear check, **LRFD [5.13.3.6.1]**.

$$\left(\frac{t_{col}}{2}+\frac{d_{v\_avg}}{2}\right)\cdot\frac{1}{12}=3.4 \qquad \text{feet}$$

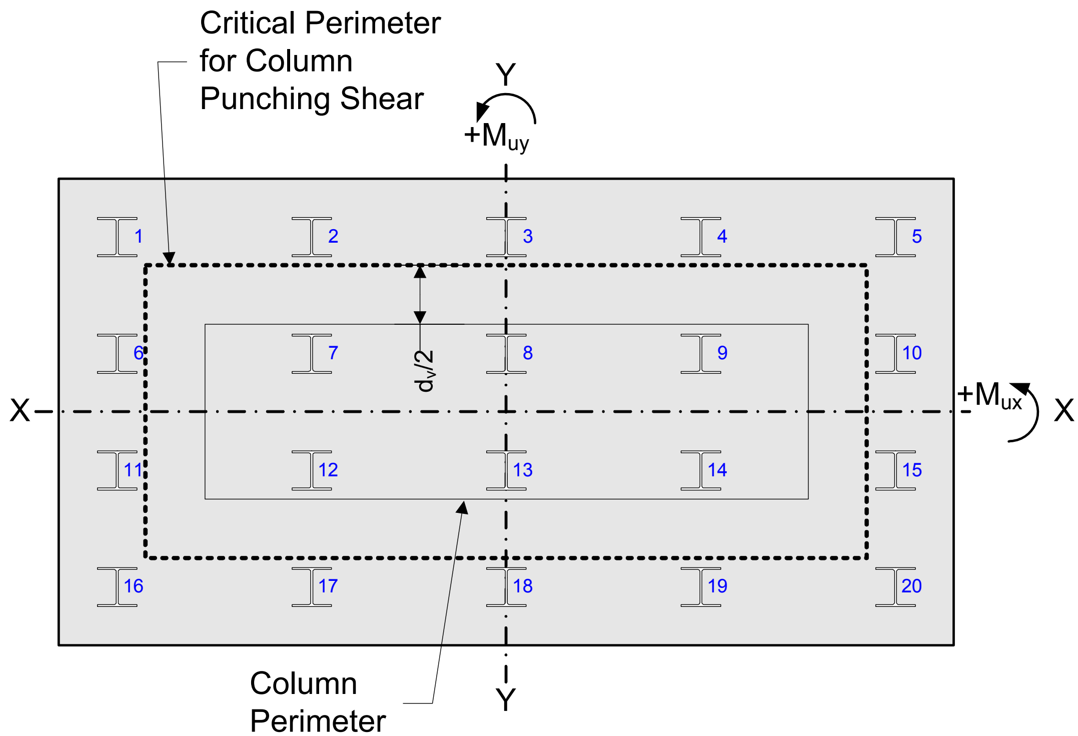


Figure E13-1.11-2
Critical Perimeter for Column Punching Shear

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):

$$V_{u\_punch} := \max\left(Pu1_{punch\_col}, Pu2_{punch\_col}, Pu3_{punch\_col}\right)$$

$\boxed{V_{u\_punch} = 2187.26}$ kips

$\boxed{V_{r\_punch} = 3263.77}$ kips

$V_{u\_punch} \le V_{r\_punch}$

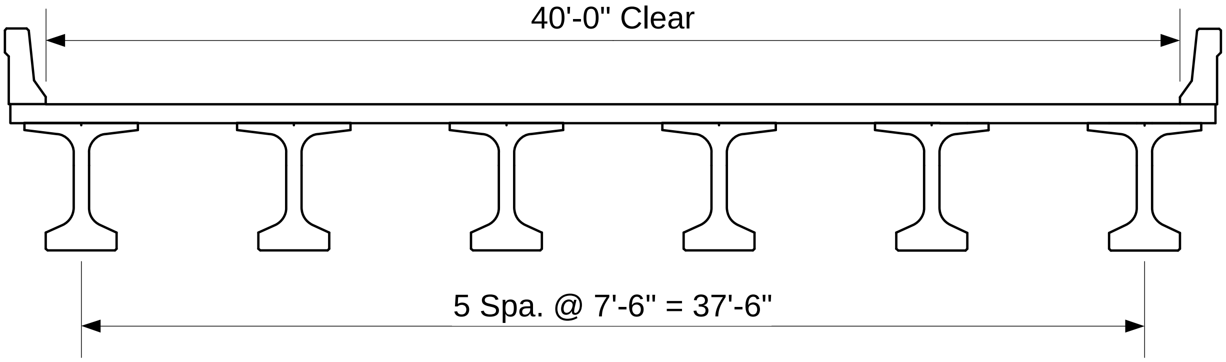
$\boxed{check = \text{"OK"}}$

For two-way action around the maximum loaded pile, the pile critical perimeter, b_o, is located a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge.

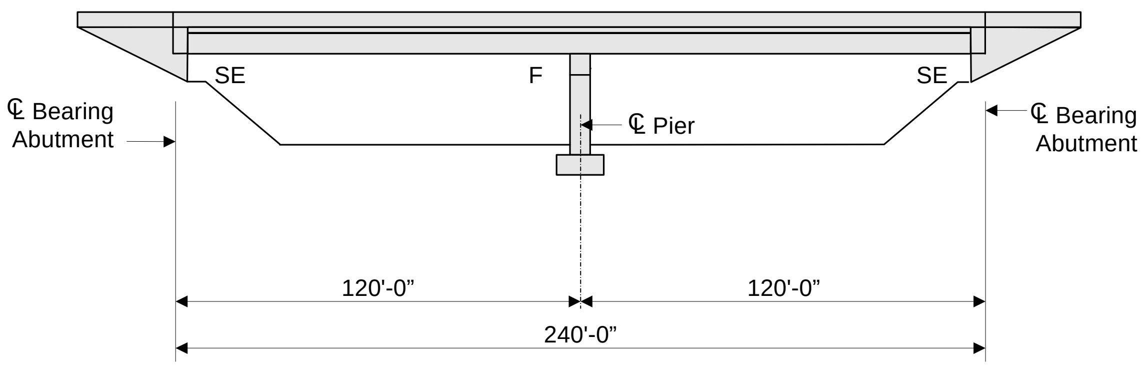
Table of Contents

E13-2 Multi-Column Pier Design Example - LRFD

2 Span Bridge, 54W, LRFD Design



This pier is designed for the superstructure as detailed in example **E19-2**. This is a two-span prestressed girder grade separation structure. Semi-expansion bearings are located at the abutments, and fixed bearings are used at the pier.



E13-2.1 Obtain Design Criteria

This multi-column pier design example is based on ***AASHTO LRFD Bridge Design Specifications, (Seventh Edition - 2015 Interim).*** The design methods presented throughout the example are meant to be the most widely used in general bridge engineering practice. Calculations are only shown for the pier cap. For example column and footing calculations, see example E13-1.

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

E13-2.1.1 Material Properties:

$w_c := 0.150$ Concrete density, kcf

$f'_c := 3.5$ Concrete 28-day compressive strength, ksi **LRFD [5.4.2.1 & Table C5.4.2.1-1]**

$f_y := 60$ Reinforcement strength, ksi **LRFD [5.4.3 & 6.10.1.7]**

$E_s := 29000$ Modulus of Elasticity of the reinforcing steel, ksi

$$E_c := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$$ **LRFD [C5.4.2.4]**

$\boxed{E_c = 3587}$ Modulus of Elasticity of the Concrete, ksi

E13-2.1.2 Reinforcing steel cover requirements (assume epoxy coated bars)

Cover dimension listed below is in accordance with **LRFD [Table 5.12.3-1].**

$Cover_{cap} := 2.5$ Concrete cover in pier cap, inches

E13-2.1.3 Relevant Superstructure Data

$L := 130$ design span length, feet

$w_b := 42.5$ out to out width of deck, feet

$w_{deck} := 40$ clear width of deck, feet

$w_p := 0.387$ weight of Wisconsin Type LF parapet, klf

$t_s := 8$ slab thickness, inches

$t_{haunch} := 4$ haunch thickness, inches

$skew := 0$ skew angle, degrees

$S := 7.5$ girder spacing, ft

$ng := 6$ number of girders

$$DOH := \frac{w_b - (ng - 1) \cdot S}{2}$$ deck overhang length $\boxed{DOH = 2.5}$ feet

$w_{tf} := 48$ width of 54W girder top flange, inches

$t_{tf} := 3$ thickness of 54W girder top flange, inches

$tf_{slope} := \frac{2.5}{20.75}$ slope of bottom surface of top flange $\boxed{tf_{slope} = 0.12}$ feet per foot

$girder_H := 54$ height of 54W girder, inches

E13-2.1.4 Select Optimum Pier Type

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. The most common pier types are single column (i.e., "hammerhead"), solid wall type, and bent type (multi-column or pile bent). For this design example, a multi-column pier was chosen.

E13-2.1.5 Select Preliminary Pier Dimensions

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

$cap_L := 41.5$ overall cap length, ft

$cap_H := 4.0$ pier cap height, ft

$cap_W := 3.5$ pier cap width, ft

$col_{spa} := 18.25$ column spacing, ft

$col_d := 3$ column depth (perpendicular to pier CL), ft

$col_w := 4$ column width (parallel to pier CL), ft

$col_h := 18$ column height, ft

$cap_{OH} := 2.5$ pier cap overhang dimension, ft

Figures E13-2.1-1 and E13-2.1-2 show the preliminary dimensions selected for this pier design example.

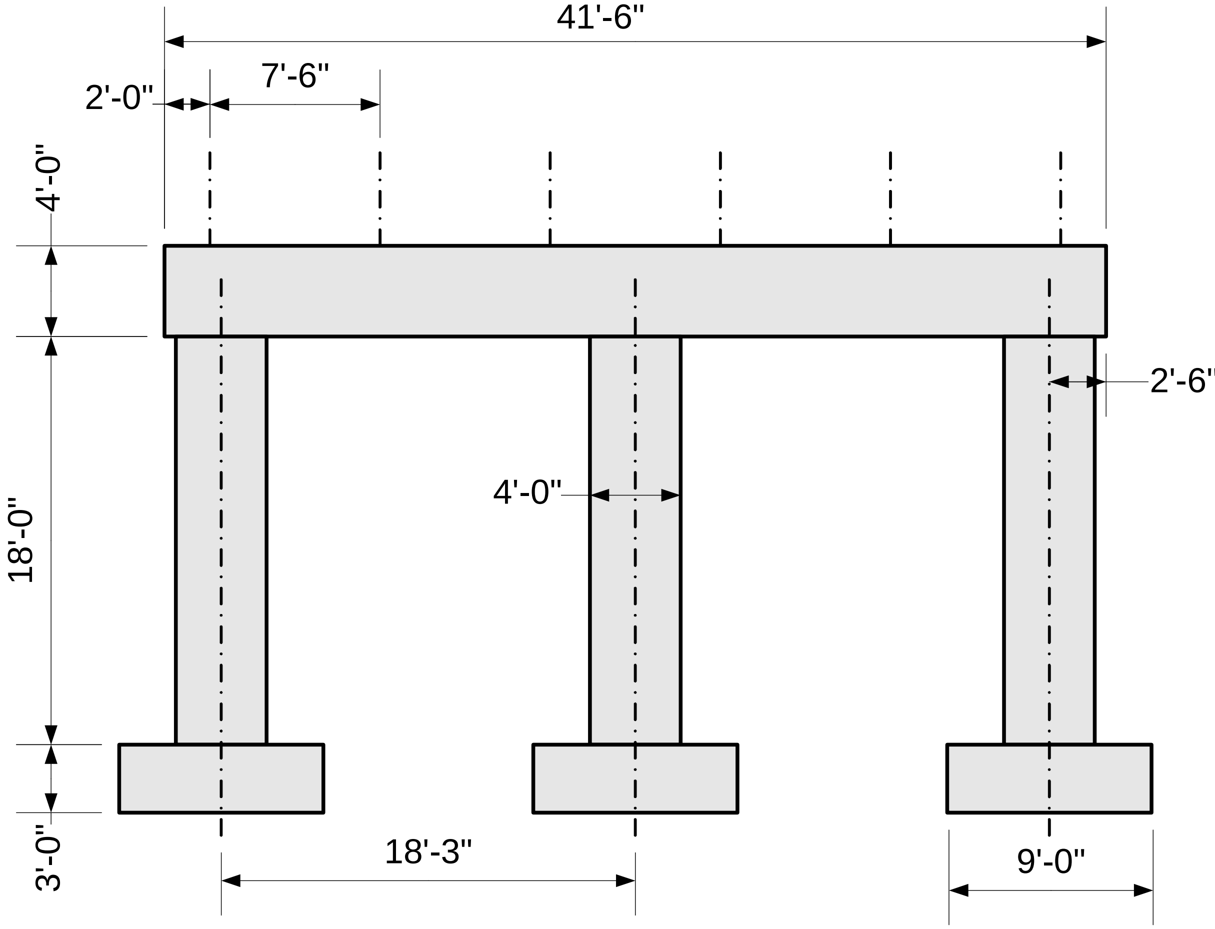


Figure E13-2.1-1
Preliminary Pier Dimensions - Front Elevation

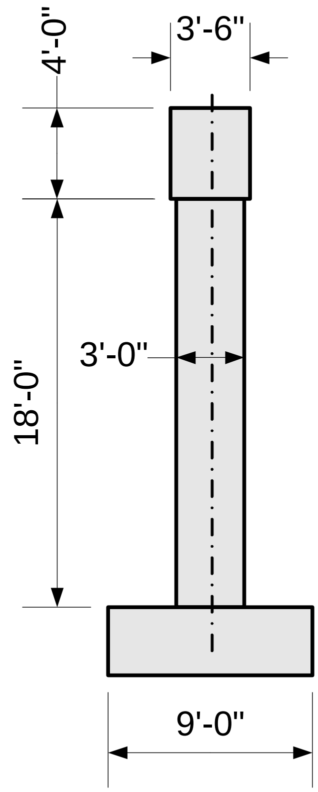


Figure E13-2.1-2
Preliminary Pier Dimensions - End Elevation

E13-2.6 Pier Cap Design

Calculate positive and negative moment requirements.

E13-2.6.1 Positive Moment Capacity Between Columns

It is assumed that there will be two layers of positive moment reinforcement. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

In accordance with **LRFD [5.10.3.1.3]** the minimum clear space between the bars in layers is one inch or the nominal diameter of the bars.

$spa_{clear} := 1.75$ in

$bar_{stirrup} := 5$ (transverse bar size)

$\boxed{Bar_D(bar_{stirrup}) = 0.63}$ in (transverse bar diameter)

$Bar_{No\_pos} := 9$

$\boxed{Bar_D(Bar_{No\_pos}) = 1.13}$ in (Assumed bar size)

$$d_e := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - Bar_D(Bar_{No\_pos}) - \frac{spa_{clear}}{2}$$

$\boxed{d_e = 42.87}$ in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the cap:

$b_w := cap_W \cdot 12$ $\boxed{b_w = 42}$ in

$\boxed{Mu_{pos} = 2372}$ kip-ft

$$R_u := \frac{Mu_{pos} \cdot 12}{\phi_f \cdot b_w \cdot d_e^{\,2}}$$ $\boxed{R_u = 0.4097}$ ksi

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.00738}$

$A_s := \rho \cdot b_w \cdot d_e$ $\boxed{A_s = 13.28}$ in^2

This requires $n_{bars\_pos} := 14$ bars. Use $n_{bars\_pos1} := 9$ bars in the bottom layer and $n_{bars\_pos2} := 5$ bars in the top layer. Check spacing requirements.

$$spa_{pos} := \frac{b_w - 2 \cdot (cover + Bar_D(bar_{stirrup})) - Bar_D(Bar_{No\_pos})}{n_{bars\_pos1} - 1}$$

$\boxed{spa_{pos} = 4.33}$ in

$clear_{spa} := spa_{pos} - Bar_D(Bar_{No\_pos})$ $\boxed{clear_{spa} = 3.2}$ in

The minimum clear spacing is equal to 1.5 times the maximum aggregate size of 1.5 inches.

$spa_{min} := 1.5 \cdot 1.5$ $\boxed{spa_{min} = 2.25}$ in

Is $spa_{min} \le clear_{spa}$? $\boxed{check = \text{"OK"}}$

$As_{prov\_pos} := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos}$ $\boxed{As_{prov\_pos} = 14}$ in^2

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi)

$$a := \frac{As_{prov\_pos} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a = 6.72}$ in

$$Mn_{pos} := As_{prov\_pos} \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12}$$ $\boxed{Mn_{pos} = 2766}$ kip-ft

$Mr_{pos} := \phi_f \cdot Mn_{pos}$ $\boxed{Mr_{pos} = 2489}$ kip-ft

$\boxed{Mu_{pos} = 2372}$ kip-ft

Is $Mu_{pos} \le Mr_{pos}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$S_{cap} := \frac{(cap_W \cdot 12) \cdot (cap_H \cdot 12)^2}{6}$$ $\boxed{S_{cap} = 16128}$ in^3

$f_r := 0.24 \cdot \sqrt{f'_c}$ $\boxed{f_r = 0.45}$ 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>

$$M_{cr} := 1.1 \cdot f_r \cdot S_{cap} \cdot \frac{1}{12}$$ $\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{pos} = 3155}$ kip-ft

Is Mr_{pos} greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{pos}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho := \dfrac{As_{prov\_pos}}{b_w \cdot d_e}$ $\boxed{\rho = 0.00778}$

$n := \text{floor}\left(\dfrac{E_s}{E_c}\right)$ $\boxed{n = 8}$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$ $\boxed{k = 0.3}$

$j := 1 - \dfrac{k}{3}$ $\boxed{j = 0.9}$

$d_c := cover + Bar_D(bar_{stirrup}) + \dfrac{Bar_D(Bar_{No\_pos})}{2}$ $\boxed{d_c = 3.69}$ in

$\boxed{Ms_{pos} = 1634}$ kip-ft

$f_s := \dfrac{Ms_{pos}}{As_{prov\_pos} \cdot j \cdot d_e} \cdot 12$ $\le 0.6\ f_y$ $\boxed{f_s = 36.24}$ ksi approx. = 0.6 f_y O.K.

The height of the section, h, is:

$h := cap_H \cdot 12$ $\boxed{h = 48}$ in

$\beta := 1 + \dfrac{d_c}{0.7 \cdot (h - d_c)}$ $\boxed{\beta = 1.12}$

$\gamma_e := 1.0$ for Class 1 exposure condition

$S_{max} := \dfrac{700 \gamma_e}{\beta \cdot f_s} - 2 \cdot d_c$ $\boxed{S_{max} = 9.89}$ in

$\boxed{spa_{pos} = 4.33}$ in

Is $spa_{pos} \le S_{max}$? $\boxed{check = \text{"OK"}}$

E13-2.6.2 Positive Moment Reinforcement Cut Off Location

Terminate the top row of bars where bottom row of reinforcement satisfies the moment diagram

$spa' := spa_{pos}$ $\boxed{spa' = 4.33}$ in

$As' := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos1}$ $\boxed{As' = 9}$ in^2

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.85}$ (for $f'_c \leq 10.0$ ksi)

$$a' := \frac{As' \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a' = 4.32}$ in

$$d_{e'} := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - \frac{Bar_D(Bar_{No\_pos})}{2}$$

$\boxed{d_{e'} = 44.31}$ in

$$M_{n'} := As' \cdot f_y \cdot \left(d_{e'} - \frac{a'}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{n'} = 1897}$ 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 $cut_{pos} := 10.7$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 1707}$ kip-ft

$\boxed{Mu_{cut1} = 1538}$ kip-ft

$\boxed{Ms_{cut1} = 1051}$ kip-ft

Is $Mu_{cut1} \leq M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{cut1} = 2045}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{cut1}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho' := \frac{As'}{b_w \cdot d_{e'}}$$ $\boxed{\rho' = 0.00484}$

$$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n$$ $\boxed{k' = 0.24}$

$$j' := 1 - \frac{k'}{3}$$ $\boxed{j' = 0.92}$

$\boxed{Ms_{cut1} = 1051}$ 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 [5.11.1.2.1]**:

$\boxed{d_{e'} = 44.31}$ in

$\boxed{15 \cdot Bar_D\left(Bar_{No\_pos}\right) = 16.92}$ in

$\boxed{\frac{col_{spa} \cdot 12}{20} = 10.95}$ in

$\boxed{BarExtend_{pos} = 44.31}$ in

The bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number $\boxed{\rightarrow 9}$ bar with spacing less than 6-inches, is:

$l_{d\_9} := 5.083$ ft

$\boxed{cut_{pos} + \frac{BarExtend_{pos}}{12} = 14.39}$

$\boxed{0.4 \cdot col_{spa} + l_{d\_9} = 12.38}$

Similar calculations show that the second layer bottom mat bars can also be terminated at a distance of 2.0 feet from the CL of the left column. At least one quarter of the bars shall be extended past the centerline of the support for continuous spans. Therefore, run the bottom layer bars to the end of the cap.

E13-2.6.3 Negative Moment Capacity at Face of Column

It is assumed that there will be one layer of negative moment reinforcement. Therefore the effective depth of the section at the pier is:

$\boxed{cover = 2.5}$ in

$\boxed{bar_{stirrup} = 5}$ (transverse bar size)

$\boxed{Bar_D(bar_{stirrup}) = 0.63}$ in (transverse bar diameter)

$Bar_{No\_neg} := 8$

$\boxed{Bar_D(Bar_{No\_neg}) = 1.00}$ in (Assumed bar size)

$$d_{e\_neg} := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - \frac{Bar_D(Bar_{No\_neg})}{2}$$

$\boxed{d_{e\_neg} = 44.38}$ in

For flexure in non-prestressed concrete, $\boxed{\phi_f = 0.9}$.
The width of the cap:

$\boxed{b_w = 42}$ in

$\boxed{Mu_{neg} = -1174}$ kip-ft

$$R_{u\_neg} := \frac{|Mu_{neg}| \cdot 12}{\phi_f \cdot b_w \cdot d_{e\_neg}^{\;2}}$$

$\boxed{R_{u\_neg} = 0.1892}$ ksi

$$\rho_{neg} := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_{u\_neg}}{0.85 \cdot f'_c}}\right)$$

$\boxed{\rho_{neg} = 0.00326}$

$$A_{s\_neg} := \rho_{neg} \cdot b_w \cdot d_{e\_neg}$$

$\boxed{A_{s\_neg} = 6.08}$ in^2

This requires $n_{bars\_neg} := 9$ bars. Check spacing requirements.

$$spa_{neg} := \frac{b_w - 2 \cdot (cover + Bar_D(bar_{stirrup})) - Bar_D(Bar_{No\_neg})}{n_{bars\_neg} - 1}$$

$\boxed{spa_{neg} = 4.34}$ in

$$clear_{spa\_neg} := spa_{neg} - Bar_D(Bar_{No\_neg})$$

$\boxed{clear_{spa\_neg} = 3.34}$ in

Is $spa_{min} \le clear_{spa\_neg}$? $\boxed{check = \text{"OK"}}$

$As_{prov\_neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg}$ $\boxed{As_{prov\_neg} = 7.07}$ in^2

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi)

$$a_{neg} := \frac{As_{prov\_neg} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a_{neg} = 3.39}$ in

$$Mn_{neg} := As_{prov\_neg} \cdot f_y \cdot \left(d_{e\_neg} - \frac{a_{neg}}{2}\right) \cdot \frac{1}{12}$$ $\boxed{Mn_{neg} = 1508}$ kip-ft

$Mr_{neg} := \phi_f \cdot Mn_{neg}$ $\boxed{Mr_{neg} = 1358}$ kip-ft

$\boxed{Mu_{neg} = 1174}$ kip-ft

Is $Mu_{neg} \le Mr_{neg}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg} = 1561}$ kip-ft

Is Mr_{neg} greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{neg}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho_{neg} := \frac{As_{prov\_neg}}{b_w \cdot d_{e\_neg}}$$ $\boxed{\rho_{neg} = 0.00379}$

$\boxed{n = 8}$

$$k_{neg} := \sqrt{(\rho_{neg} \cdot n)^2 + 2 \cdot \rho_{neg} \cdot n} - \rho_{neg} \cdot n$$ $\boxed{k_{neg} = 0.22}$

$$j_{neg} := 1 - \frac{k_{neg}}{3}$$ $\boxed{j_{neg} = 0.93}$

$$d_{c\_neg} := cover + Bar_D(bar_{stirrup}) + \frac{Bar_D(Bar_{No\_neg})}{2}$$ $\boxed{d_{c\_neg} = 3.63}$ in

$\boxed{Ms_{neg} = 844}$ kip-ft

$$f_{s\_neg} := \frac{Ms_{neg}}{As_{prov\_neg} \cdot j_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \le 0.6\, f_y$$ $f_{s\_neg} = 34.8$ ksi $\le 0.6\, f_y$ O.K.

The height of the section, h, is: $\boxed{h = 48}$ in

$$\beta_{neg} := 1 + \frac{d_{c\_neg}}{0.7 \cdot (h - d_{c\_neg})}$$ $\boxed{\beta_{neg} = 1.12}$

$\gamma_e := 1.0$ for Class 1 exposure condition

$$S_{max\_neg} := \frac{700\gamma_e}{\beta_{neg} \cdot f_{s\_neg}} - 2 \cdot d_{c\_neg}$$ $\boxed{S_{max\_neg} = 10.76}$ in

$\boxed{spa_{neg} = 4.34}$ in

Is $spa_{neg} \le S_{max\_neg}$? $\boxed{check = "OK"}$

E13-2.6.4 Negative Moment Reinforcement Cut Off Location

Cut 4 bars where the remaining 5 bars satisfy the moment diagram.

$n_{bars\_neg'} := 5$

$spa'_{neg} := spa_{neg} \cdot 2$ $\boxed{spa'_{neg} = 8.69}$ in

$As'_{neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg'}$ $\boxed{As'_{neg} = 3.93}$ in^2

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi)

$$a'_{neg} := \frac{As'_{neg} \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a'_{neg} = 1.89}$ in

$\boxed{d_{e\_neg} = 44.38}$ in

$$M_{n'\_neg} := As'_{neg} \cdot f_y \cdot \left(d_{e\_neg} - \frac{a'_{neg}}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{n'\_neg} = 853}$ kip-ft

$M_{r'\_neg} := \phi_f \cdot M_{n'\_neg}$ $\boxed{M_{r'\_neg} = 768}$ kip-ft

Based on the moment diagram, try locating the cut off at $cut_{neg} := 15.3$ feet from the CL of the left column. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'\_neg} = 768}$ kip-ft

$\boxed{Mu_{neg\_cut} = 577}$ kip-ft

$\boxed{Ms_{neg\_cut} = 381}$ kip-ft

Is $Mu_{neg\_cut} \le M_{r'\_neg}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 664}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg\_cut} = 767}$ kip-ft

Is $M_{r'\_neg}$ greater than the lesser value of M_{cr} and $1.33 \cdot Mu_{neg\_cut}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho'_{neg} := \frac{As'_{neg}}{b_w \cdot d_{e\_neg}} \qquad \boxed{\rho'_{neg} = 0.00211}$$

$$k'_{neg} := \sqrt{\left(\rho'_{neg} \cdot n\right)^2 + 2 \cdot \rho'_{neg} \cdot n} - \rho'_{neg} \cdot n \qquad \boxed{k'_{neg} = 0.17}$$

$$j'_{neg} := 1 - \frac{k'_{neg}}{3} \qquad \boxed{j'_{neg} = 0.94}$$

$\boxed{Ms_{neg\_cut} = 381}$ kip-ft

$$f_{s'\_neg} := \frac{Ms_{neg\_cut}}{As'_{neg} \cdot j'_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \le 0.6\ f_y \qquad \boxed{f_{s'\_neg} = 27.79} \text{ ksi} \quad \le 0.6\ f_y \text{ O.K.}$$

$\boxed{\beta_{neg} = 1.12}$

$\boxed{\gamma_e = 1}$

$$S_{max'\_neg} := \frac{700 \gamma_e}{\beta_{neg} \cdot f_{s'\_neg}} - 2 \cdot d_{c\_neg} \qquad \boxed{S_{max'\_neg} = 15.30} \text{ in}$$

$\boxed{spa'_{neg} = 8.69}$ in

Is $spa'_{neg} \le S_{max'\_neg}$? $\boxed{check = \text{"OK"}}$

The bars shall be extended past this cut off point for a distance not less than the following, **LRFD [5.11.1.2.3]**:

$d_{e\_neg} = 44.38$ 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 [5.8.3.3]**:

$$V_{n1} = V_c + V_s + V_p$$

$$V_{n2} = 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$$

V_c , the shear resistance due to concrete (kip), is calculated as follows:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

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, β = 2.0, **LRFD [5.8.3.4.1]**.

$V_c := 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\boxed{V_c = 211.94}$ kips

V_s , the shear resistance due to steel (kips), is calculated as follows:

$$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s}$$

Where:

s = spacing of stirrups (in)
θ = angle of inclination of diagonal compressive stresses (deg)
α = angle of inclination of transverse reinforcement to longitudinal axis (deg)

$s := 5$ in

$\theta := 45deg$ for non prestress members

$\alpha := 90deg$ for vertical stirrups

A_v = (# of stirrup legs)(area of stirrup)

$\boxed{bar_{stirrup} = 5}$

StirrupConfig := "Triple"

$\boxed{stirrup_{legs} = 6}$

$A_v := stirrup_{legs} \cdot \left(Bar_A\left(bar_{stirrup}\right)\right)$ $\boxed{A_v = 1.84}$ in^2

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s}$$ $\boxed{V_s = 942.74}$ kips

V_p , the component of the effective prestressing force in the direction of the applied shear:

$V_p := 0$ for non prestressed members

V_n is the lesser of:

$V_{n1} := V_c + V_s + V_p$ $\boxed{V_{n1} = 1154.67}$ kips

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $\boxed{V_{n2} = 1568.41}$ kips

Therefore, use: $\boxed{V_n = 1154.67}$ kips

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 1039.2}$ kips

$\boxed{Vu = 978.82}$ kips

Is $Vu \le V_r$? $\boxed{check = \text{"OK"}}$

Check the Minimum Transverse Reinforcement, **LRFD [5.8.2.5]**

Required area of transverse steel:

$$Av_{min} := 0.0316 \cdot \frac{\sqrt{f'_c} \cdot b_v \cdot s}{f_y}$$

$\boxed{Av_{min} = 0.21}$ in^2

$\boxed{A_v = 1.84}$ in^2

Is $Av_{min} \le A_v$ (provided area of steel)? $\boxed{check = "OK"}$

Check the Maximum Spacing of the Transverse Reinforcement, **LRFD [5.8.2.7]**

If v_u<0.125f'_c, then: $s_{max} := 0.8 \cdot d_v \le 24in$

If v_u > or = 0.125f'_c, then: $s_{max} := 0.4 \cdot d_v \le 12in$

The shear stress on the concrete, v_u, is taken to be:

$$v_u := \frac{Vu}{\phi_v \cdot b_v \cdot d_v}$$

$\boxed{v_u = 0.61}$ ksi

$\boxed{0.125 \cdot f'_c = 0.44}$ ksi

$\boxed{s_{max} = 12}$ in

$\boxed{s = 5}$ in

Is the spacing provided $s \le s_{max}$? $\boxed{check = "OK"}$

Similar calculations are used to determine the required stirrup spacing for the remainder of the cap.

$\boxed{s_2 = 12}$ in

$\boxed{StirrupConfig_2 = "Double"}$

$\boxed{Vu2 = 276}$ kips

$\boxed{V_{r\_2} = 408.94}$ kips

$\boxed{s_3 = 6}$ in

$\boxed{StirrupConfig_3 = "Double"}$

$\boxed{Vu3 = 560}$ kips

$\boxed{V_{r\_3} = 627.13}$ kips

It should be noted that the required stirrup spacing is typically provided for a distance equal to the cap depth past the CL of the girder. Consideration should also be given to minimize the number of stirrup spacing changes where practical. These procedures result in additional capacity in the pier cap that is often beneficial for potential future rehabilitation work on the structure.

E13-2.6.6 Temperature and Shrinkage Steel

Temperature and shrinkage steel shall be provided on each face and in each direction as calculated below. **LRFD [5.10.8]**

$\boxed{cap_W = 3.5}$ ft

$\boxed{cap_H = 4}$ ft

$b := cap_W \cdot 12$ $\boxed{b = 42}$ in

$\boxed{h = 48}$ in

$$As_{ts} := \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y}$$ $\boxed{As_{ts} = 0.24}$ in^2/ft in each face

Is the area required As_{ts} between 0.11 and 0.60 in^2 per foot? $\boxed{check = \text{"OK"}}$

Use number 5 bars at one foot spacing: $\boxed{Bar_A(5) = 0.31}$ in^2/ft in each face

E13-2.6.7 Skin Reinforcement

If the effective depth, d_e, of the reinforced concrete member exceeds 3 ft., longitudinal skin reinforcement is uniformly distributed along both side faces of the component for a distance of de/2 nearest the flexural tension reinforcement, **LRFD [5.7.3.4]**. The area of skin reinforcement (in^2/ft of height) on each side of the face is required to satisfy:

$A_{sk} \ge 0.012(d_e - 30)$ and $A_{sk} \cdot \left(\frac{d_e}{2 \cdot 12}\right)$ need not exceed $(A_s / 4)$

Where: (For positive moment region)

A_{sk} = area of skin reinforcement (in^2/ft)

A_s = area of tensile reinforcement (in^2) $\boxed{A_s = 13.28}$ in^2

d_e = flexural depth taken as the distance from the compression face to the centroid of the steel, positive moment region (in) $\boxed{d_e = 42.87}$ in

$A_{sk1} := 0.012 \cdot (d_e - 30)$ $\boxed{A_{sk1} = 0.15}$ in^2/ft

$$A_{sk1} := A_{sk1} \cdot \left(\frac{d_e}{2 \cdot 12}\right)$$ $\boxed{A_{sk1} = 0.28}$ in^2

$$A_{sk2} := \frac{A_s}{4}$$ $\boxed{A_{sk2} = 3.32}$ in^2

$A_{face} := \min(A_{sk1}, A_{sk2})$ (area req'd. per face within de/2 from tension reinf.) $\boxed{A_{face} = 0.28}$ in^2

$$spa\_max_{sk} := \min\left(\frac{d_e}{6}, 12\right)$$ $\boxed{spa\_max_{sk} = 7.15}$ in

Use number 5 bars at 6" spacing: $\boxed{Bar_A(5) \cdot 2 = 0.61}$ in^2 > A_{face}
(provides 2 bars within de/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

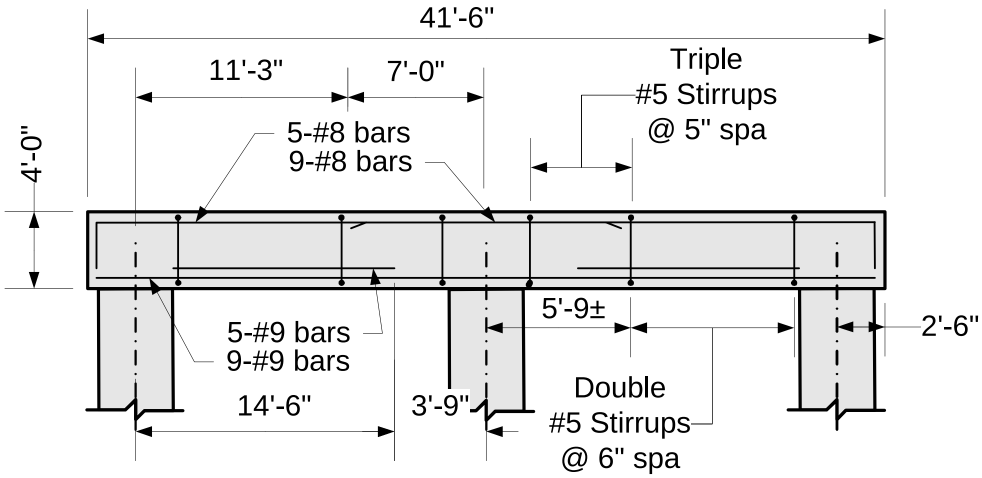


Figure E13-2.7-1
Cap Reinforcement - Elevation View

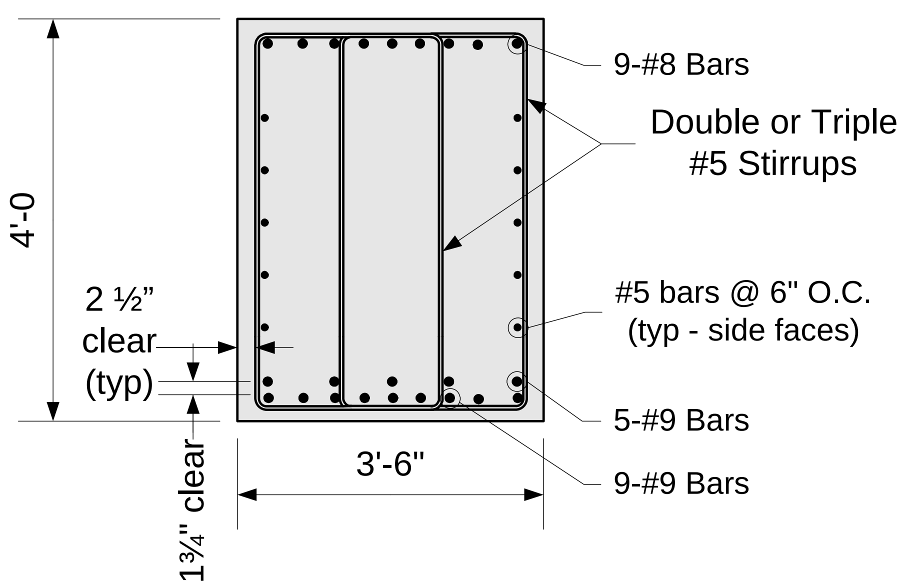


Figure E13-2.7-2
Cap Reinforcement - Section View

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Table of Contents

14.1 Introduction

Retaining walls are used to provide lateral resistance for a mass of earth or other material to accommodate a transportation facility. These walls are used in a variety of applications including right-of-way restrictions, protection of existing structures that must remain in place, grade separations, new highway embankment construction, roadway widening, stabilization of slopes, protection of environmentally sensitive areas, staging, and temporary support including excavation or underwater construction support, etc.

Several types of retaining wall systems are available to retain earth and meet specific project requirements. Many of these wall systems are proprietary wall systems while others are non-proprietary or design-build in Wisconsin. The wall selection criteria and design policies presented in this chapter are to ensure consistency of standards and applications used throughout WisDOT projects.

WisDOT policy item:

Retaining walls (such as MSE walls with precast concrete panel facing) that are susceptible to damage from vehicular impact shall be protected by a roadway barrier.

14.1.1 Wall Development Process

Overall, the wall development process requires an iterative collaboration between WisDOT Regions, Structures Design Section, Geotechnical Engineering Unit and WisDOT Consultants.

Retaining wall development is described in Section 11-55-5 of the *Facilities Development Manual*. WisDOT Regional staff determines the need for permanent retaining walls on highway projects. A wall number is assigned as per criteria discussed in 14.1.1.1 of this chapter. The Regional staff prepares a Structures Survey Report (SSR) that includes a preliminary evaluation of wall type, location, and height including a preliminary layout plan.

Based on the SSR, a Geotechnical site investigation (see Chapter 10 – Geotechnical Investigation) may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Geotechnical Engineering Unit can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results. These Geotechnical recommendations are presented in a Site Investigation Report.

The SSR is sent to the wall designer (Structures Design Section or WisDOT's Consultant) for wall selection, design and contract plan preparation. Based on the wall selection criteria discussed in 14.3, either a proprietary or a non-proprietary wall system is selected.

Proprietary walls, as defined in 14.2, are pre-approved by the WisDOT's Bureau of Structures. Preapproval process for the proprietary walls is explained in 14.16. The structural design, internal and final external stability of proprietary wall systems are the responsibility of the supplier/contractor. The design and shop drawing computations of the proprietary wall systems are also reviewed by the Bureau of Structures in accordance with the plans and special provisions. The preliminary external stability, overall stability and settlement computations of these walls are performed by the Geotechnical Engineering Unit or the WisDOT's Consultant in the project design phase. Design and shop drawings must be accepted by the Bureau of Structures prior to start of the construction. Design of all temporary walls is the responsibility of the contractor.

Non-proprietary retaining walls are designed by WisDOT or its Consultant. The internal stability and the structural design of such walls are performed by the Structures Design Section or WisDOT's Consultant. The external and overall stability is performed by the Geotechnical Engineering Unit or Geotechnical Engineer of record.

The final contract plans of retaining walls include final plans, details, special provisions, contract requirements, and cost estimate for construction. The Subsurface Exploration sheet depicting the soil borings is part of the final contract plans.

The wall types and wall selection criteria to be used in wall selection are discussed in 14.2 and 14.3 of this chapter respectively. General design concepts of a retaining wall system are discussed in 14.4. Design criteria for specific wall systems are discussed in sections 14.5 thru 14.11. The plan preparation process is briefly described in Chapter 2 – General and Chapter 6 – Plan Preparation. The contract documents and contract requirements are discussed in 14.14 and 14.15 respectively.

For further information related to wall selection, design, approval process, pre-approval and review of proprietary wall systems please contact Structures Design Section of the Bureau of Structures at 608-266-8489. For questions pertaining to geotechnical analyses and geotechnical investigations please contact the Geotechnical Engineering Unit at 608-246-7940.

14.1.1.1 Wall Numbering System

Permanent retaining walls that are designed for a design life of 75 years or more should be identified by a wall number, R-XX-XXX, as assigned by the Region unless otherwise specified below. For a continuous wall consisting of various wall types, the numbering system should include unit numbers so that the numbering appears as R-XX-XXX-001, R-XX-XXX-002, and so on. The first two digits represent the county the wall is located in and the next set(s) of digits represent the undivided wall.

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

- <u>Proprietary retaining walls</u> (e.g., modular block MSE walls)
    - MSE walls having a maximum height of less than 5.5 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be "minor retaining walls" and do not require an R number. Refer to *FDM 11-55-5.2* for more information.

  - Modular block gravity walls having a maximum height of less than 4.0 ft. measured from the bottom of wall or top of leveling pad to top of wall are deemed to be “minor retaining walls” and do not require an R number. Refer to *FDM 11-55-5.2* for more information.

- <u>Non-proprietary walls</u> (e.g., sheet pile walls):

  - Walls having an exposed height of less than 5.5 ft. measured from the plan ground line to top of wall may require an R number based on specific project features. Designer to contact the Bureau of Structures region liaison for more information.

Cast-in-place walls being utilized strictly as bridge abutment or box culvert wings do not require R numbers as they are considered part of the structure.

14.2 Wall Types

Retaining walls can be divided into many categories as discussed below.

Conventional Walls

Retaining walls can be divided into gravity, semi-gravity, and non-gravity cantilever or anchored walls. A brief description of these walls is presented in 14.2.1 and 14.2.2 respectively.

Miscellaneous types of walls including multi-tiered walls, and hybrid or composite walls are also used by combining the wall types mentioned in the previous paragraph. These walls are used only under special project requirements. These walls are briefly discussed in 14.2.3, but the design requirements of these walls will not be presented in this chapter. In addition, some walls are also used for temporary shoring and discussed briefly in 14.2.4.

Permanent or Temporary Walls

All walls can be divided into permanent or temporary walls, depending on project application. Permanent walls have a typical designed life of 75 years. The temporary walls are designed for a service life of 3 years, or the intended project duration, whichever is greater. Temporary wall systems have less restrictive requirements for construction, material and aesthetics.

Fill Walls or Cut Walls

A retaining wall can also be classified as a fill wall, or a cut wall. This description is based on the nature of the earthwork required to construct the wall. If the roadway cross-sections (which include the wall) indicate that existing earth/soil must be removed (excavated) to install the wall, it is considered a ‘cut’ wall. If the roadway cross-sections indicate that earth fill will be placed behind the wall, with little excavation, the wall is considered a ‘fill’ wall. Sometimes wall construction requires nearly equal combinations of earth excavation and earth fill, leading to the nomenclature of a ‘cut/fill’ wall.

Bottom-up or Top-down Constructed Walls

This wall classification method refers to the method in which a wall is constructed. If a wall is constructed from the bottom of the wall, upward to the top, it is considered a bottom-up type of wall. Examples of this include CIP cantilever, MSE and modular block walls. Bottom-up walls are generally the most cost effective type. If a wall is constructed downward, from the top of the wall to the bottom, it is considered a top-down type of wall. This generally requires the insertion of some type of wall support member below the existing ground, and then excavation in front of the wall to the bottom of the exposed face. Examples of this include soil nail, soldier pile, cantilever sheet pile and anchored sheet pile walls. These walls are generally used when excavation room is limited.

Proprietary or Non-Proprietary

Some retaining walls have prefabricated modules or components that are proprietary in nature. Based on the use of proprietary components, walls can be divided into the categories of proprietary and non-proprietary wall systems as defined in 14.1.1.

A proprietary retaining wall system is considered as a patented or trademarked retaining wall system or a wall system comprised of elements/components that are protected by a trade name, brand name, or patent and are designed and supported by the manufacturer. MSE walls, modular block gravity walls, bin, and crib walls are considered proprietary walls because these walls have components which are either patented or have trademarks.

Proprietary walls require preapproval and appropriate special provisions. The preapproval requirements are discussed in 14.16 of this chapter. Proprietary walls also have special design requirements for the structural components, and are discussed in further detail within each specific wall design section. Most MSE, modular block, bin or crib walls require pre-approval and/or special provisions.

A non-proprietary retaining wall is fully designed and detailed by the designer or may be design-build. A non-proprietary retaining wall system may contain proprietary elements or components as well as non-proprietary elements and components. CIP cantilever walls, rock walls, soil nail walls and non-gravity walls fall under this category.

Wall classification is shown in Table 14.2-1 and is based on wall type, project function category, and method of construction.

14.2.1 Gravity Walls

Gravity walls are considered externally stabilized walls as these walls use self weight to resist lateral pressures due to earth and water. Gravity walls are generally subdivided into mass gravity, semi-gravity, modular gravity, mechanically stabilized reinforced earth (MSE), and in-situ reinforced earth wall (soil nailing) categories. A schematic diagram of the various types of gravity walls is included in Figure 14.2-1.

14.2.1.1 Mass Gravity Walls

A mass gravity wall is an externally stabilized, cast-in-place rigid gravity wall, generally trapezoidal in shape. The construction of these walls requires a large quantity of materials so these are rarely used except for low height walls less than 8.0 feet. These walls mainly rely on self weight to resist external pressures and their construction is staged as bottom up construction, mostly in fill or cut/fill situations.

14.2.1.2 Semi-Gravity Walls

Semi-gravity walls resist external forces by the combined action of self weight, weight of soil above footing and the flexural resistance of the wall components. A cast-in-place (CIP) concrete cantilever wall is an example and consists of a reinforced concrete stem and a base footing. These walls are non-proprietary.

Cantilever walls are best suited for use in areas exhibiting good bearing material. When bearing or settlement is a problem, these walls can be founded on piles or foundation improvement may be necessary. The use of piles significantly increases the cost of these walls. Walls exceeding 28 feet in height are provided with counter-forts or buttress slabs. Construction of these walls is staged as bottom-up construction and mostly constructed in fill situations. Cantilever walls are more suited where MSE walls are not feasible, although these walls are generally costlier than MSE walls.

14.2.1.3 Modular Gravity Walls

Modular walls are also known as externally stabilized gravity walls as these walls resist external forces by utilizing self weight. Modular walls have prefabricated modules/components which are considered proprietary. The construction is bottom-up construction mostly used in fill situations.

14.2.1.3.1 Modular Block Gravity Walls

Modular block concrete facings are used without soil reinforcement to function as an externally stabilized gravity wall. The modular blocks are prefabricated dry cast or wet cast concrete blocks and the blocks are stacked vertically or slightly battered to resist external forces. The concrete blocks are either solid concrete or hollow core concrete blocks. The hollow core concrete blocks are filled with crushed aggregates or sand. Modular block gravity walls are limited to a maximum design height of 8 feet under optimum site geometry and soils conditions, but site conditions generally dictate the need for MSE walls when design heights are greater than 5.5 feet. Walls with a maximum height of less than 4 feet are deemed as “minor retaining walls” and do not require an R number. Refer to *FDM 11-55-5.2* for more information. The modular blocks are proprietary and vary in sizes.

14.2.1.3.2 Prefabricated Bin, Crib and Gabion Walls

<u>Bin Walls</u>: Concrete and metal bin walls are built of adjoining open or closed faced bins and then filled with soil/rocks. Each metal bin is comprised of individual members bolted together. The concrete bin wall is comprised of prefabricated interlocking concrete modules. These wall systems are proprietary wall systems.

<u>Crib Walls</u>: Crib walls are constructed of interlocking prefabricated units of reinforced or unreinforced concrete or timber elements. Each crib is comprised of longitudinal and transverse members. Each unit is filled with free draining material. These wall systems are proprietary wall systems.

<u>Gabion Walls</u>**:** Gabion walls are constructed of steel wire baskets filled with selected rock fragments and tied together. Gabions walls are flexible, free draining and easy to construct. These wall systems are proprietary wall systems. Maximum heights are normally less than 21 feet. These walls are desirable where equipment access is limited. The wires used for constructing gabions baskets must be designed with adequate corrosion protection.

14.2.1.4 Rock Walls

Rock walls are also known as 'Rockery Walls'. These types of gravity walls are built by stacking locally available large stones or boulders into a trapezoid shape. These walls are highly flexible and height of these walls is generally limited to approximately 8.0 feet. A layer of gravel and geotextile is commonly used between the stones and the retained soil. These walls can be designed using the *FHWA Rockery Design and Construction Guideline*.

14.2.1.5 Mechanically Stabilized Earth (MSE) Walls:

Mechanically Stabilized Earth (MSE) walls include a selected soil mass reinforced with metallic or geosynthetic reinforcement. The soil reinforcement is connected to a facing element to prevent the reinforced soil from sloughing. Construction of these walls is staged as bottom-up construction. These can be constructed in cut and fill situations, but are better suited to fill sites. MSE walls are normally used for wall heights between 10 to 40 feet. A brief description of various types of MSE walls is given below:

<u>Precast Concrete Panel MSE Walls</u>: These types of walls employ a metallic strip or wire grid reinforcement connected to precast concrete panels to reinforce a selected soil mass. The concrete panels are usually 5'x5' or 5'x10' size panels. These walls are proprietary wall systems.

<u>Modular Block Facing MSE Wall</u>: Prefabricated modular concrete block walls consist of almost vertically stacked concrete modular blocks and the soil reinforcement is secured between the blocks at predetermined levels. Metallic strips or geogrids are generally used as soil reinforcement to reinforce the selected soil mass. Concrete blocks are either solid or hollow core blocks, and must meet freeze/thaw requirements. The hollow core blocks are filled with aggregates or sand. These types of walls are proprietary wall systems.

<u>Geotextile/Geogrids/Welded Wire Faced MSE Walls</u>: These types of MSE walls consist of compacted soil layers reinforced with continuous or semi-continuous geotextile, geogrid or welded wire around the overlying reinforcement. The wall facing is formed by wrapping each layer of reinforcement around the overlying layer of backfill and re-embedding the free end into the backfill. These types of walls are used for temporary or permanent applications. Permanent facings include shotcrete, gunite, galvanized welded wire mesh, cast-in-place concrete or prefabricated concrete panels.

14.2.1.6 Soil Nail Walls

Soil nail walls are internally stabilized cut walls that use in-situ reinforcement for resisting earth pressures. The large diameter rebars (generally #10 or greater) are typically used for the reinforcement. The construction of soil nail walls is staged top-down and soil nails are installed after each stage of excavation. Shotcrete can be applied as a facing. The facing of a soil nail wall is typically covered with vertical drainage strips located over the nail then covered with shotcrete. Soil nail walls are used for temporary or permanent construction. Specialty contractors are required when constructing these walls. Soil nail walls have been installed to heights of 60.0 feet or more but there have only been a limited number of soil nail walls constructed on WisDOT projects.

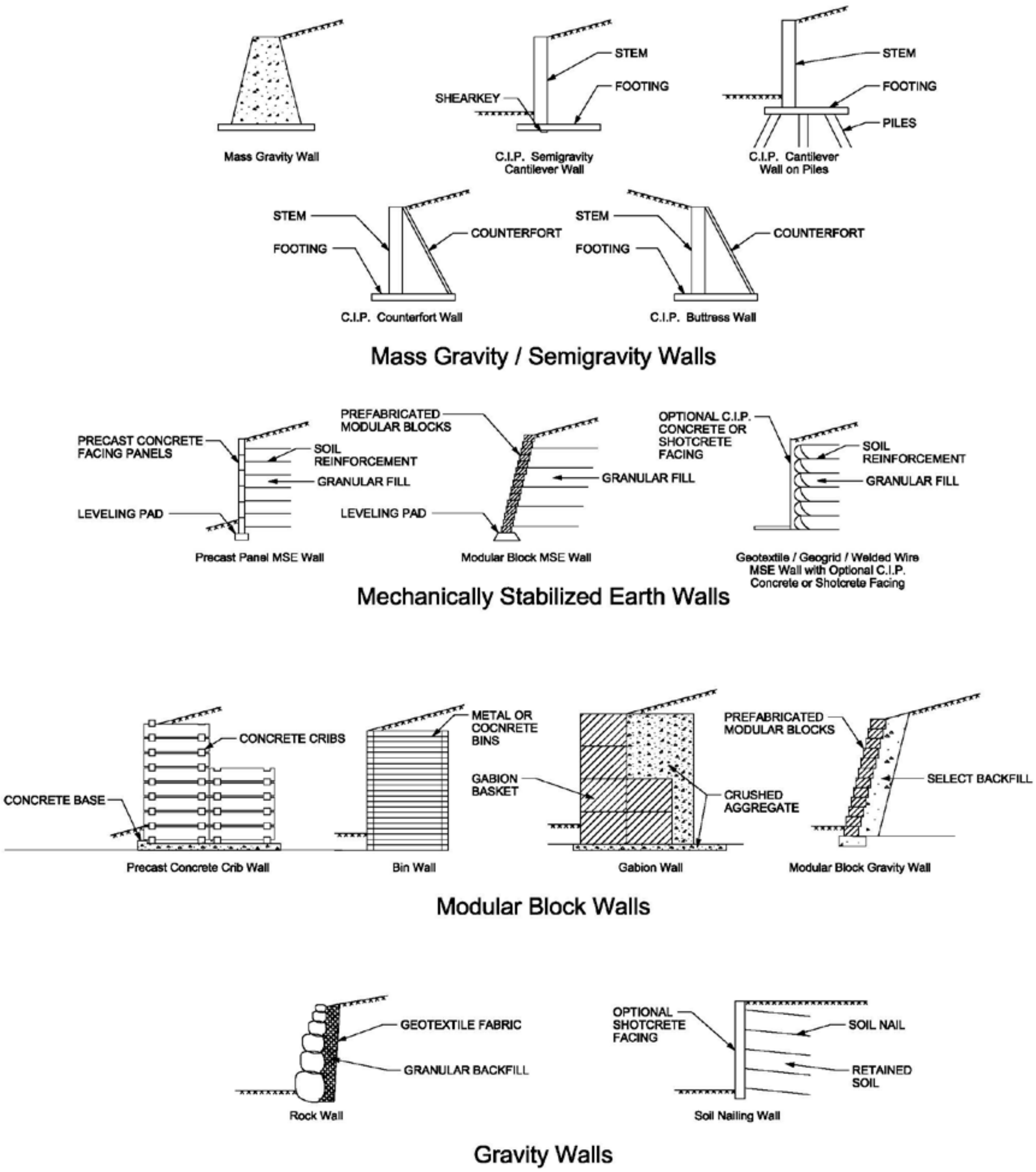


Figure 14.2-1
Gravity Walls

14.2.2 Non-Gravity Walls

Non-gravity walls are classified into cantilever and anchored wall categories. These walls are considered as externally stabilized walls and generally used in cut situations. The walls include sheet pile, soldier pile, tangent and secant pile type with or without anchors. Figure 14.2-2 shows common types of non-gravity walls.

14.2.2.1 Cantilever Walls

These types of walls derive lateral resistance through embedment of vertical elements into natural ground and the flexure resistance of the structural members. They are used where excavation support is needed in shallow cut situations.

<u>Cantilever Sheet Pile Walls</u>: Cantilever sheet pile walls consist of interlocking steel panels, driven into the ground to form a continuous sheet pile wall. The sheet piles resist the lateral earth pressure utilizing the passive resistance in front of the wall and the flexural resistance of the sheet pile. Most sheet pile walls are less than 15 feet in height.

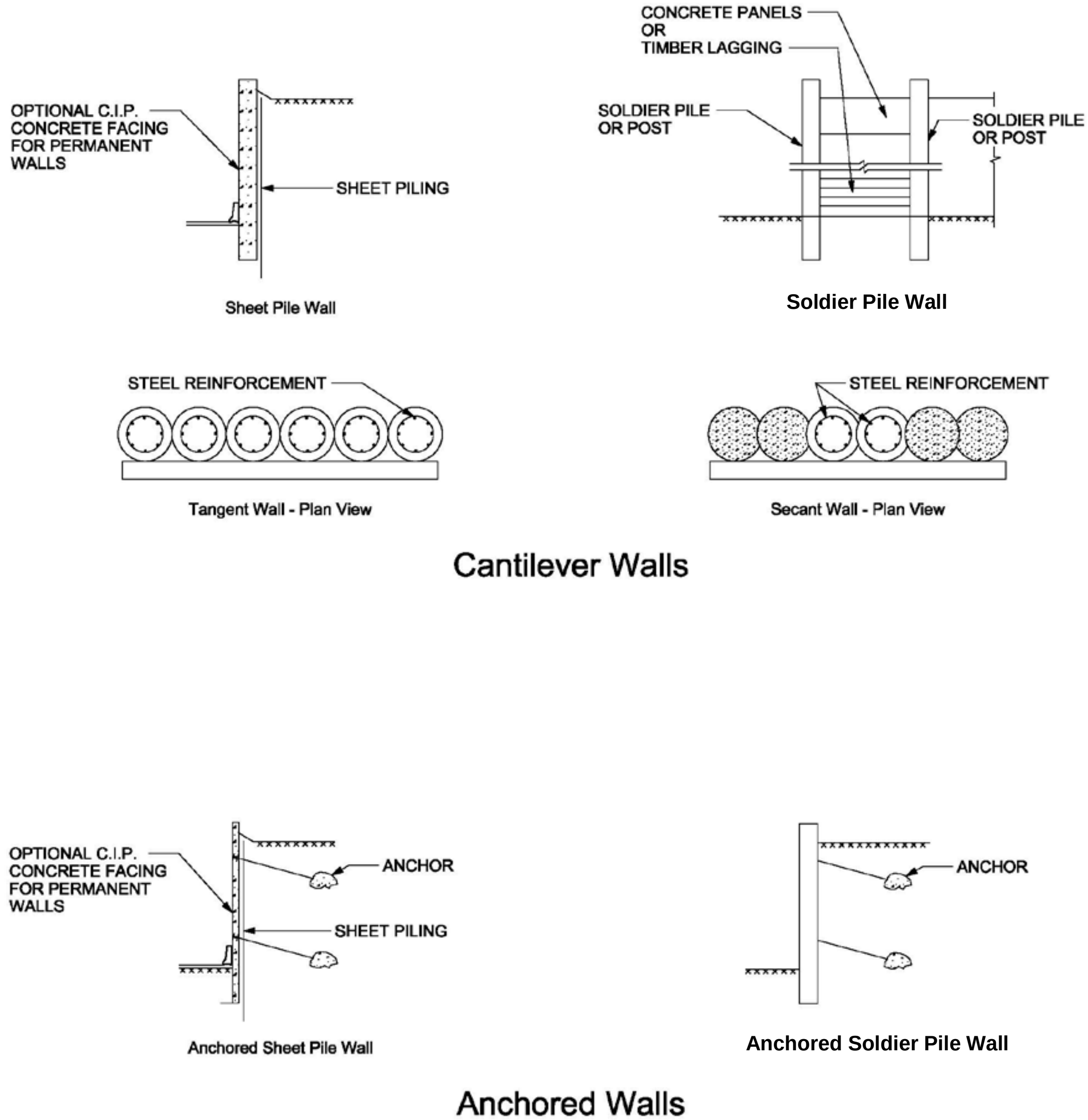
<u>Soldier Pile Walls</u>: A soldier pile wall derives lateral resistance and moment capacity through embedment of vertical members (soldier piles) into natural ground usually in cut situations. The vertical elements (usually H piles) may be drilled or driven steel or concrete members. The soil behind the wall is retained by lagging. The lagging may be steel, wood, or concrete. For permanent walls, wall facings are usually constructed of either cast-in-place concrete or precast concrete panels (prestressed, if needed) that extend between vertical elements. Solider pile walls that use precast panels and H piles are also known as post-and-panel walls. Soldier pile walls can also be constructed from the bottom-up. These walls should be considered when minimizing disturbance to the site is critical, such as environmental and/or construction procedures. Soldier pile walls are also suitable for sites where rock is encountered near the surface, since holes for the piles can be drilled/prebored into the rock.

<u>Tangent and Secant Pile Walls</u>: A tangent pile wall consists of a single row of drilled shafts (bored piles) installed in the ground. Each pile touches the adjacent pile tangentially. The concrete piles are reinforced using a single steel beam or a steel reinforcement cage. A secant wall, similar to a tangent pile wall, consists of overlapping adjacent piles. All piles generally contain reinforcement, although alternating reinforced piles may be necessary. Secant and tangent wall systems are used to hold earth and water where water tightness is important, and lowering of the water table is not desirable. To improve wall water tightness, additional details can used to minimize water seepage.

14.2.2.2 Anchored Walls

Anchored walls are externally stabilized non-gravity cut walls. Anchored walls are essentially the same as cantilever walls except that these walls utilize anchors (tiebacks) to extend the wall heights beyond the design limit of the cantilever walls. These walls require less toe embedment than cantilever walls.

These walls derive lateral resistance by embedment of vertical wall elements into firm ground and by anchorages. Most commonly used anchored walls are anchored sheet pile walls and soldier pile walls. Tangent and secant walls can also be anchored with tie backs and used as anchored walls. The anchors can be attached to the walls by tie rods, bars or wire tendons. The anchoring device is generally a deadman, screw-type, or grouted tieback anchor. Anchored walls can be built to significant heights using multiple rows of anchors.



<u>Figure 14.2-2</u>
Non-Gravity Walls

14.2.3 Tiered and Hybrid Wall Systems

A tiered wall system is a series of two or more walls, with each wall set back from the underlying walls. The upper wall exerts an additional surcharge on the lower lying wall and requires special design attention. The design of these walls has not been discussed in this chapter. Hybrids wall systems combine wall components from two or more different wall systems and provide an alternative to a single type of wall used in cut or fill locations. These types of walls require special design attention as components of these walls require different magnitudes of deformation to develop loading resistance. The design of such walls will be on a case-by-case basis, and is not discussed in this chapter.

Some examples of tiered and hybrid walls systems are shown in Figure 14.2-3.

14.2.4 Temporary Shoring

Temporary shoring is used to protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Shoring should not be required nor paid for when used primarily for the convenience of the contractor. Temporary shoring is designed by the contractor and may consist of a wall system, or some other type of support. MSE walls with flexible facings and sheet pile walls are commonly used for temporary shoring.

14.2.5 Wall Classification Chart

A wall classification chart has been developed and shown as Table 14.2-1.

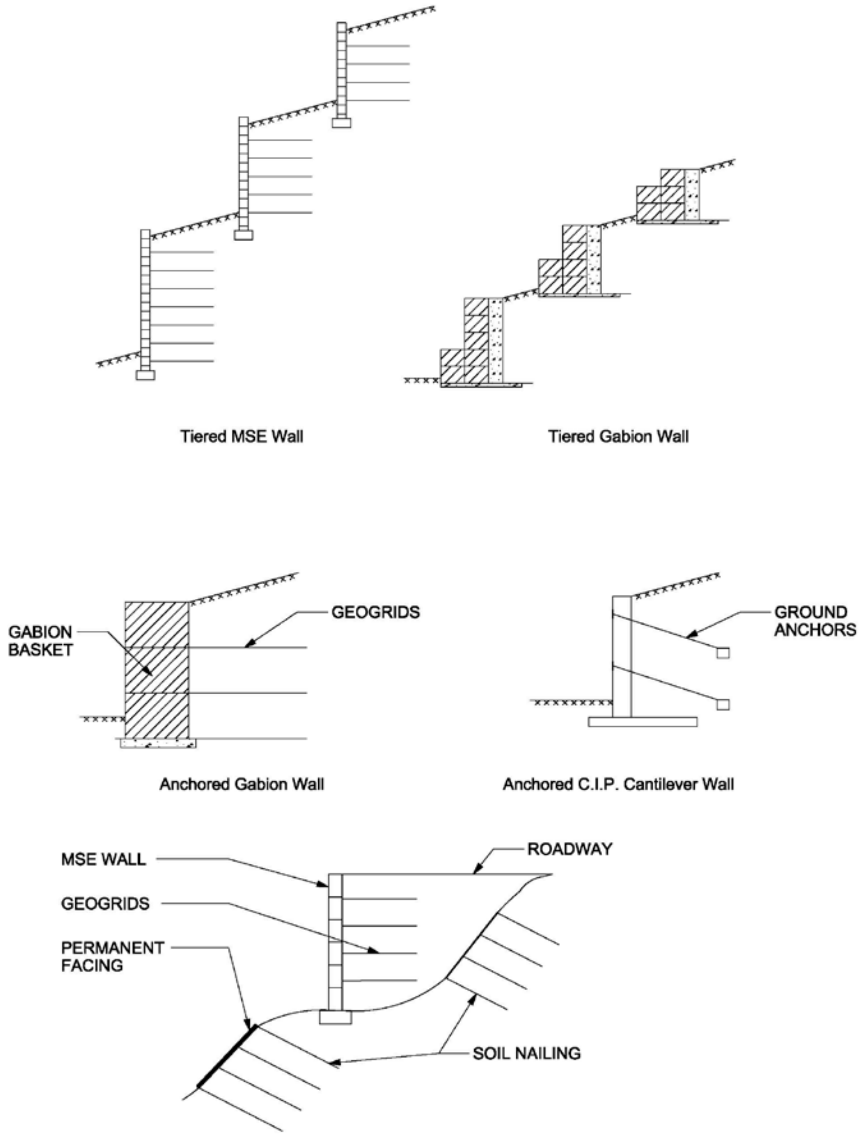


Figure 14.2-3
Tiered & Hybrid Wall Systems

Wall Category	Wall Sub-Category	Wall Type	Typical Construction Concept	Proprietary
Gravity	Mass Gravity	CIP Concrete Gravity	Bottom Up (Fill)	No
	Semi-Gravity	CIP Concrete Cantilever	Bottom Up (Fill)	No
	Reinforced Earth	MSE Walls: • Precast Panels • Modular Blocks • Geogrid/ Geo-textile/Wire- Faced	Bottom Up (Fill)	Yes
	Modular Gravity	Modular Blocks, Gabion, Bin, Crib	Bottom Up (Fill)	Yes
	In-situ Reinforced	Soil Nailing	Top Down (Cut)	No
Non-Gravity	Cantilever	Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut) /Bottom Up (Fill)	No
	Anchored	Anchored Sheet Pile, Soldier Pile, Tangent/Secant	Top Down (Cut)	No

Table 14.2-1
Wall Classification

14.3 Wall Selection Criteria

14.3.1 General

The objective of selecting a wall system is to determine an appropriate wall system that is practical to construct, structurally sound, economic, aesthetically pleasing, environmentally consistent with the surroundings, and has minimal maintenance problems.

With the development of many new wall systems, designers have the choice of selecting many feasible wall systems that can be constructed on a given highway project. Designers are encouraged to evaluate several feasible wall systems for a particular project where wall systems can be economically constructed. After consideration of various wall types, a single type should be selected for final analyses and design. Wall designers must consider the general design concepts described in section 14.4 and specific wall design requirements described in 14.5 thru 14.11 of this chapter, and key wall selection factors discussed in this section.

In general, selection of a wall system should include, but not limited to the key factors described in this section for consideration when generating a list of acceptable retaining wall systems for a given site.

14.3.1.1 Project Category

The designer must determine if the wall system is permanent or temporary.

14.3.1.2 Cut vs. Fill Application

Due to construction techniques and base width requirements for stability, some wall types are better suited for cut sections where as others are suited for fill or fill/cut situations. The key considerations are the amount of excavation or shoring, overall wall height, proximity of wall to other structures, and right-of-way width available. The site geometry should be evaluated to define site constraints. These constraints will generally dictate if fill, fill/cut or cut walls are required.

Cut Walls

Cut walls are generally constructed from the top down and used for both temporary and permanent applications. Cantilever sheet pile walls are suitable for shallower cuts. If a deeper cut is required to be retained, a key question is to determine the availability of right-of-way (ROW). Subsurface conditions such as shallow bedrock also enter into considerations of cut walls. Anchored walls, soil nail walls, and anchored soldier pile walls may be suitable for deeper cuts although these walls require either a larger permanent easement or permanent ROW.

Fill walls

Walls constructed in fill locations are typically used for permanent construction and may require large ROW to meet the base width requirements. The necessary fill material may be required to be granular in nature. These walls use bottom up construction and have typical cost effective ranges. Surface conditions must also be considered. For instance, if soft compressible soils are present, walls that can tolerate larger settlements and movements must be considered. MSE walls are generally more economical for fill locations than CIP cantilever walls.

Cut/fill Walls

CIP cantilever and prefabricated modular walls are most suitable in cut/fill situations as the walls are built from bottom up, have narrower base widths and these walls do not rely on soil reinforcement techniques to provide stability. These types of walls are suitable for both cut or fill situations.

14.3.1.3 Site Characteristics

Site characterization should be performed, as appropriate, to provide the necessary information for the design and construction of retaining wall systems. The objective of this characterization is to determine composition and subsurface soil/rock conditions, define engineering properties of foundation material and retained soils, establish groundwater conditions, determine the corrosion potential of the water, identify any discontinuities or geotechnical issues such as poor bearing capacity, large settlement potential, and/or any other design and construction problems.

Site characterization mainly includes subsurface investigations and analyses. WisDOT's Geotechnical Engineering Unit generally completes the investigation and analyses for all in-house wall design work.

14.3.1.4 Miscellaneous Design Considerations

Other key factors that may influence wall selection include height limitations for specific systems, limit of wall radius on horizontal alignment, and whether the wall is a component of an abutment.

Foundation conditions that may govern the wall selection are bearing capacity, allowable lateral and vertical movements, tolerable settlement and differential movement of retaining wall systems being designed, susceptibility to scour or undermining due to seepage, and long-term maintenance.

14.3.1.5 Right of Way Considerations

Availability of ROW at a site may influence the selection of wall type. When a very narrow ROW is available, a sheet pile wall may be suitable to support an excavation. In other cases, when walls with tiebacks or soil reinforcement are considered, a relatively large ROW may be required to meet wall requirements. Availability of vertical operating space may influence wall selection where piling installation is required and there is not enough room to operate driving equipment.

Section 11-55-5 of the FDM describes the ROW requirement for retaining walls. It requires that all segments of a retaining wall should be under the control of WisDOT. No improvements or utility construction should be allowed in the ROW area of the retaining wall systems.

14.3.1.6 Utilities and Other Conflicts

Feasibility of some wall systems may be influenced by the presence of utilities and buried structures. MSE, soil nailing and anchored walls commonly have conflict with the presence of utilities or buried underground structures. MSE walls should not be used where utilities must stay in the reinforcement zone.

14.3.1.7 Aesthetics

In addition to being functional and economical, the walls should be aesthetically pleasing. Wall aesthetics may influence selection of a particular wall system. However, the aesthetic treatment should complement the retaining wall and not disrupt the functionality or selection of wall type. All permanent walls should be designed with due considerations to the wall aesthetics. Each wall site must be investigated individually for aesthetic needs. Temporary walls should generally be designed with little consideration to aesthetics. Chapter 4 - Aesthetics presents structures aesthetic requirements.

14.3.1.8 Constructability Considerations

Availability of construction materials, site accessibility, equipment availability, form work and temporary shoring, dewatering requirements, labor considerations, complicated alignment changes, scheduling consideration, speed of construction, construction staging/phasing and maintaining traffic during construction are some of the important key factors when evaluating the constructability of each wall system for a specific project site.

In addition, it should also be ensured that the temporary excavation slopes used for wall construction are stable as per site conditions and meet all safety requirements laid by Occupation and Safety Health Administration (OSHA).

14.3.1.9 Environmental Considerations

Selection of a retaining wall system is influenced by its potential environmental impact during and after construction. Some of the environmental concerns during construction may include excavation and disposal of contaminated material at the project site, large quantity of water, corrosive nature of soil/water, vibration impacts, noise abatement and pile driving constraints.

14.3.1.10 Cost

Cost of a retaining wall system is influenced by many factors that must be considered while estimating preliminary costs. The components that influence cost include excavation, structure, procurement of additional easement or ROW, drainage, disposal of unsuitable material, traffic maintenance etc. Maintenance cost also affects overall cost of a retaining wall system. The retaining walls that have least structural cost may not be the most economical walls. Wall selection should be based on overall cost. When feasible, MSE Walls and modular block gravity walls generally cost less than other wall types.

14.3.1.11 Mandates by Other Agencies

In certain project locations, other agency mandates may limit the types of wall systems considered.

14.3.1.12 Requests made by the Public

A Public Interest Finding could dictate the wall system to be used on a specific project.

14.3.1.13 Railing

For safety reasons most walls will require a protective railing. The railing will usually be located behind the wall. The roadway designer will generally determine whether a pedestrian or non-pedestrian railing is required and what aesthetic considerations are needed.

14.3.1.14 Traffic barrier

A traffic barrier should be installed if vehicles, bicycles, or pedestrians are likely to be present on top of the wall. The roadway designer generally determines the need for a traffic barrier.

14.3.2 Wall Selection Guide Charts

Table 14.3-1 and Table 14.3-2 summarize the characteristics for the various wall types that are normally considered during the wall selection process. The tables also present some of the advantages, disadvantages, cost effective height range and other key selection factors. A wall designer can use these tables and the general wall selection criteria discussed in 14.3.1 as a guide. Designers are encouraged to contact the Structures Design Section if they have any questions relating to wall selection for their project.

Wall Type	Temp.	Perm.	Cost Effective Height (ft)	Req'd. ROW	Advantages	Disadvantages
CIP Concrete Gravity		√	3 - 10	0.5H - 0.7H	• Durable • Meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time
CIP Concrete Cantilever		√	6 - 28	0.4H - 0.7H	• Durable meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time & deeper embedment
Reinforced CIP Counterfort		√	26 - 40	0.4H - 0.7H	• Durable • Meets aesthetic requirement • Requires small back fill quantity	• High cost • May need deep foundation • Longer const. time & deeper embedment
Modular Block Gravity		√	3 - 8	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Height limitations
Metal Bin		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or special equipment	• Difficult to make height adjustment in the field
Concrete Crib		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Difficult to make height adjustment in the field
Gabion		√	6 - 20	0.4H - 0.7H	• Does not require skilled labor or specialized equipment	• Need large stone quantities • Significant labor
MSE Wall (precast concrete panel with steel reinforcement)		√	10 - 35	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (modular block and geo-synthetic reinforcement)		√	6 - 22	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill
MSE Wall (geotextile/geogrid/ welded wire facing)	√	√	6 - 35	0.7H - 1.0H	• Does not require skilled labor or specialized equipment	• Requires use of select backfill

Table 14.3-1
Wall Selection Chart for Gravity Walls

Wall Type	Temp.	Perm.	Cost Effective Height (ft)	Req'd. ROW	Water Tightness	Advantages	Disadvantages
Sheet Pile	√	√	6 - 15	Minimal	Fair	• Rapid construction • Readily available	• Deep foundation may be needed • Longer construction time
Soldier Pile	√	√	6 - 28	0.2H - 0.5H	Poor	• Easy construction • Readily available	• High cost • Deep foundation may be needed • Longer construction time
Tangent Pile		√	20 - 60	0.4H - 0.7H	Fair/Poor	• Adaptable to irregular layout • Can control wall stiffness	• High cost • Deep foundation may be needed • Longer construction
Secant Pile		√	14 - 60	0.4H - 0.7H	Fair	• Adaptable to irregular layout • Can control wall stiffness	• Difficult to make height adjustment in the field • High cost
Anchored	√	√	15 - 35	0.4H - 0.7H	Fair/Poor	• Rapid construction	• Difficult to make height adjustment in the field
Soil Nail	√	√	6 - 20	0.4H - 0.7H	Fair	• Option for top-down	• Cannot be used in all soil types • Cannot be used below water table • Significant labor

Table 14.3-2
Wall Selection Chart for Non-Gravity Walls

14.4 General Design Concepts

This section covers the general design standards and criteria to be used for the design of temporary and permanent gravity and non-gravity walls including proprietary and non-proprietary wall systems.

The design criteria for tiered walls that retain other walls or hybrid walls systems requiring special design are not covered specifically in this section.

14.4.1 General Design Steps

The design of wall systems should follow a systematic process applicable for all wall systems and summarized below:

1. Basic Project Requirement: This includes determination of wall alignment, wall geometry, wall function, aesthetic, and project constraints (e.g. right of way, easement during construction, environment, utilities etc) as part of the wall development process described in 14.1.

2. Wall Selection: Select wall type based on step 1 and the wall section criteria discussed in 14.3.

3. Geotechnical Investigation: Subsurface investigation and analyses should be performed in accordance with 14.4.4 and Chapter 10 - Geotechnical Investigation to develop foundation and fill material design strength parameters and foundation bearing capacity. Note: this work generally requires preliminary checks performed in step 7, based on steps 4 thru 6.

4. Wall Loading: Determine all applicable loads likely to act on the wall as discussed in 14.4.5.3.

5. Initial Wall Sizing: This step requires initial sizing of various wall components and establishing wall batter which is wall specific and described under each specific wall designs discussed in 14.5 thru 14.13.

6. Wall Design Requirements: Design wall systems using design standards and service life criteria and the *AASHTO Load and Resistance Factor Design* (*AASHTO LRFD*) requirements discussed in 14.4.1 and 14.4.2.

7. Perform external stability, overall stability, and wall movement checks discussed in 14.4.7. These checks will be wall specific and generally performed by the Geotechnical Engineer of record. The stability checks should be performed using the performance limits, load combinations, and the load/resistance factors per *AASHTO LRFD* requirements described in 14.4.5.5 and 14.4.5.6 respectively.

8. Perform internal stability and structural design of the individual wall components and miscellaneous components. These computations are performed by the Designer for non-proprietary walls. For proprietary walls, internal stability is the responsibility of the contractor/supplier after letting.

14.4.5.6 Resistance Requirements and Resistance Factors

The wall components shall be proportioned by the appropriate methods so that the factored resistance as shown in **LRFD [1.3.2.1-1]** is no less than the factored loads, and satisfy criteria in accordance with **LRFD [11.5.4]** and **LRFD [11.6] thru [11.11]**. The factored resistance R_r is computed as follows: $R_r = \phi R_n$

Where

R_r = Factored resistance

R_n = Nominal resistance recommended in the Geotechnical Report

ϕ = Resistance factor

The resistance factors shall be selected in accordance with **LRFD [Tables 10.5.5.2.2-1, 10.5.5.2.3-1, 10.5.5.2.4-1, 11.5.6.1]**. Commonly used resistance factors for retaining walls are presented in Table 14.4-2.

14.4.6 Material Properties

The unit weight and strength properties of retained earth and foundation soil/rock (γ_f) are supplied in the geotechnical report and should be used for design purposes. Unless otherwise noted or recommended by the Designer or Geotechnical Engineer of record, the following material properties shall be assumed for the design and analysis if the selected backfill, concrete, and steel conforms to the WisDOT's *Standard Construction Specifications*:

Granular Backfill Soil Properties:

Internal Friction angle of backfill ϕ_f = 30 degrees

Backfill cohesion c = 0 psf

Unit Weight γ_f = 120 pcf

Concrete:

Compressive strength, f'_c at 28 days = 3500 psi

Unit Weight = 150 pcf

Steel reinforcement:

Yield strength f_y = 60,000 psi

Modulus of elasticity E_s = 29,000 ksi

Wall-Type and Condition		Resistance Factors
Mechanically Stabilized Earth Walls, Gravity Walls, and Semi-Gravity		
Bearing resistance	• Gravity & Semi-gravity • MSE	0.55 0.65
Sliding		1.00
Tensile resistance of metallic reinforcement and connectors	Strip reinforcement • Static loading Grid reinforcement • Static loading	0.75 0.65
Tensile resistance of geo-synthetic reinforcements and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
Prefabricated Modular Walls		
Bearing		**LRFD [10.5]**
Sliding		**LRFD [10.5]**
Passive resistance		**LRFD [10.5]**
Non-Gravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		**LRFD [10.5]**
Passive resistance of vertical elements		0.75
Pullout resistance of anchors	• Cohesionless soils • Cohesive soils • Rock	0.65 0.70 0.50
Pullout resistance of anchors	• Where proof tests are conducted	1.00
Tensile resistance of anchor tendons	• Mild steel • High strength steel	0.90 0.80
Flexural capacity of vertical elements		0.90

Table 14.4-2
Resistance Factors
Source **LRFD [Table 11.5.7-1]**

14.4.7.2 Wall Settlement

Retaining walls shall be designed for the effects of total and differential foundation settlement at the Service I limit state, in accordance with **LRFD [11.5.2]** and 11.2. Maximum tolerable retaining wall total and differential foundation settlements are controlled largely by the potential for cosmetic and/or structural damage to facing elements, copings, barrier, guardrail, signs, pavements, utilities, structure foundations, and other highway appurtenances supported on or near the retaining wall.

14.4.7.2.1 Settlement Guidelines

The following table provides guidance for maximum tolerable vertical and total differential Settlement for various retaining wall types where Δh is the total settlement in inches and

Wall Type	Total Settlement Δh in inches	Total Differential Settlement Δh1:L (in/in)
CIP semi-gravity cantilever walls	1-2	1:500
MSE walls with large pre-cast panel facing (panel front face area >30ft^2)	1-2	1:500
MSE walls with small pre-cast panel facing (panel front face area <30ft^2)	1-2	1:300
MSE walls with full-height cast-in-panel facing	1-2	1:500
MSE walls with modular block facing	2-4	1:200
MSE walls with geotextile /welded-wire facing	4-8	1:50-1:60
Modular block gravity walls	1-2	1:300
Concrete Crib walls	1-2	1:500
Bin walls	2-4	1:200
Gabion walls	4-6	1:50
Non-gravity cantilever and anchored walls	1-2.5	----

Table 14.4-3
Maximum Tolerable Settlement Guidelines for Retaining Walls

∆h1:L is the ratio of the difference in total vertical settlement between two points along the wall base to the horizontal distance between the two points(L). It should be noted that the tolerance provided in Table 14.4-3 are for guidance purposes only. More stringent tolerances may be required to meet project-specific requirements.

14.4.7.3 Overall Stability

Overall stability of the walls shall be checked at the Service I limit state using appropriate load combinations and resistance factors in accordance with **LRFD [11.6.2.3].** The stability is evaluated using limit state equilibrium methods. The Modified Bishop, Janbu or Spencer method may be used for the analysis. The analyses shall investigate all potential internal, compound and overall shear failure surfaces that penetrate the wall, wall face, bench, back-cut, backfill, and/or foundation zone. The overall stability check is performed by the Geotechnical Engineering Unit for WISDOT designed walls.

14.4.7.4 Internal Stability

Internal stability checks including anchor pullout or soil reinforcement failure and/or structural failure checks are also required as applicable for different wall systems. As an example, see Figure 14.4-11 for internal stability failure of MSE walls. Internal stability checks must be performed at Strength Limits in accordance with **LRFD [11.5.3].**

14.4.7.5 Wall Embedment

The minimum wall footing embedment shall be 1.5 ft below the lowest adjacent grade in front of the wall.

The embedment depth of most wall footings should be established below the depths the foundation soil/rock could be weakened due to the effect of freeze thaw, shrink-swell, scour, scour, erosion, erosion, construction excavation. The potential scour elevation shall be established in accordance with 11.2.2.1.1 of the Bridge Manual.

The final footing embedment depth shall be based on the required geotechnical bearing resistance, wall settlement limitations, and all internal, external, and overall (global) wall stability requirements in *AASHTO LRFD* and the *Bridge Manual*.

14.4.7.6 Wall Subsurface Drainage

Retaining wall drainage is necessary to prevent hydrostatic pressure and frost pressure. Inadequate wall sub-drainage can cause premature deterioration, reduced stability and collapse or failure of a retaining wall.

A properly designed wall sub-drainage system is required to control potentially damaging hydrostatic pressures and seepage forces behind and around a wall. A redundancy in the sub-drainage system is required where subsurface drainage is critical for maintaining retaining wall stability. This is accomplished using a pervious granular fill behind the wall.

Pipe underdrain must be provided to drain this fill. Therefore, "Pipe Underdrain Wrapped 6-Inch" is required behind all gravity retaining walls where seepage should be relieved. Gabion walls do not require a pipe drain system as these are porous due to rock fill. It is best to place the pipe underdrain at the top of the wall footing elevation. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain could be placed higher.

Pipe underdrains and weep holes may discharge water during freezing temperatures. In urban areas, this may create a problem due to the accumulation of flow and ice on sidewalks. Consideration should be given to connect the pipe underdrain to the storm sewer system.

14.4.7.7 Scour

The probable depth of scour shall be determined by subsurface exploration and hydraulic studies if the wall is located in flood prone areas. Refer to 11.2.2.1.1 for guidance related to scour vulnerability and design of walls. All walls with shallow foundations shall be founded below the scour elevation.

14.4.7.8 Corrosion

All metallic components of WISDOT retaining wall systems subjected to corrosion, should be designed to last through the designed life of the walls. Corrosion protection should be designed in accordance with the criteria given in **LRFD [11.10.6].** In addition, **LRFD [11.8.7] thru [11.10]** also include design guidance for corrosion protection on non-gravity cantilever walls, anchored walls and MSE walls respectively.

14.4.7.9 Utilities

Walls that have or may have future utilities in the backfill should minimize the use of soil reinforcement. MSE, soil nail, and anchored walls commonly have conflicts with utilities and should not be used when utilities must remain in or below the reinforced soil zone unless there is no other wall option. Utilities that are encapsulated by wall reinforcement may not be accessible for replacement or maintenance. Utility agreements should specifically address future access if wall reinforcing will affect access.

14.4.7.10 Guardrail and Barrier

Guardrail and barrier shall meet the requirements of the Chapter 30 - Railings, *Facilities Development Manual,* Standard Plans, and *AASHTO LRFD*. In no case shall guardrail be placed through MSE wall or reinforced slope soil reinforcement closer than 3 ft from the back of the wall facing elements. Furthermore, the guard rail posts shall be installed through the soil reinforcement in a manner that prevents ripping, damage and distortion of the soil reinforcement. In addition, the soil reinforcement shall be designed to account for the reduced cross-section resulting from the guardrail post holes.

14.5 Cast-In-Place Concrete Cantilever Walls

14.5.1 General

A cast-in-place, reinforced concrete cantilever wall is a semi-gravity wall that consists of a base slab or footing from which a vertical wall or stem extends upward. Reinforcement is provided in both members to supply resistance to bending. These walls are generally founded on good bearing material. Cantilever walls shall not be used without pile support if the foundation stratum is prone to excessive vertical or differential settlement, unless subgrade improvements are made. Cantilever walls are typically designed to a height of 28 feet. For heights exceeding 28 feet, consideration should be given to providing a counterfort. Design of counterfort CIP walls is not covered in this chapter.

CIP cantilever walls shall be designed in accordance with *AASHTO LRFD,* design concepts presented in 14.4 and the *WisDOT Standard Specifications* including the special provisions.

14.5.2 Design Procedure for Cast-in-Place Concrete Cantilever Walls

The CIP wall shall be designed to resist lateral pressure caused by supported earth, surcharge loads and water in accordance with **LRFD [11.6].** The external stability, settlement, and overall stability shall be evaluated at the appropriate load limit states in accordance with **LRFD [11.5.5],** to resist anticipated failure mechanism. The structural components mainly stem and footing should be designed to resist flexural resistance in accordance with **LRFD [11.6.3].**

Figure 14.5-1 shows possible external stability failure and deep seated rotational failure mechanisms of CIP cantilever walls that must be investigated as part of the stability check.

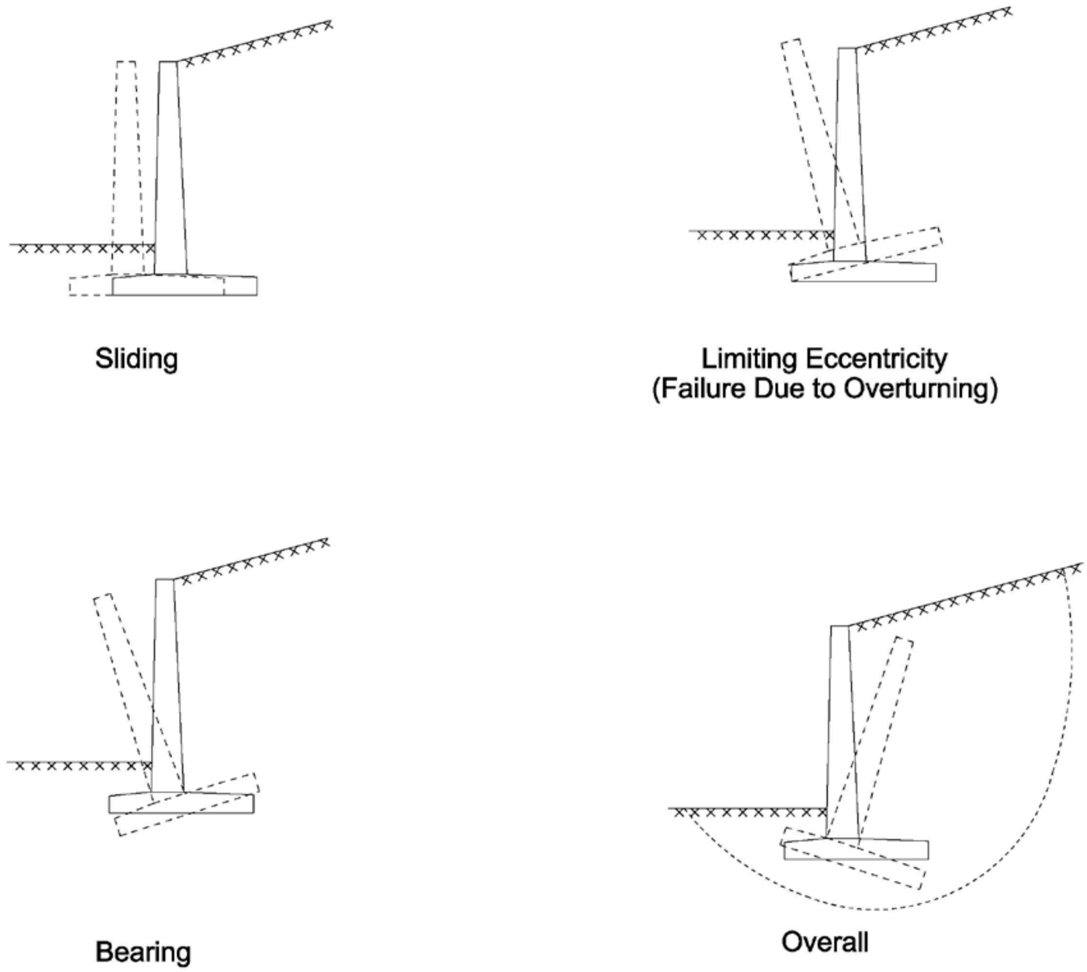


Figure 14.5-1
CIP Semi-Gravity Wall Failure Mechanism

14.5.2.1 Design Steps

The general design steps discussed in 14.4.1 shall be followed for the wall design. These steps as applicable for CIP cantilever walls are summarized below.

1. Establish project requirements including wall height, geometry and wall location as discussed in 14.1 of this chapter.

2. Perform Geotechnical investigation

3. Develop soil strength parameters

4. Determine preliminary sizing for external stability evaluation

5. Determine applicable unfactored or nominal loads

6. Evaluate factored loads for all appropriate limit states

7. Perform stability check to evaluate bearing resistance, eccentricity, and sliding as part of external stability

8. Estimate wall settlement and lateral wall movement to meet guidelines stated in Table 14.4-3.

9. Check overall stability and revise design, if necessary, by repeating steps 4 to 8.

It is assumed that steps 1, 2 and 3 have been performed prior to starting the design process.

14.5.3 Preliminary Sizing

A preliminary design can be performed using the following guideline.

1. The wall height and alignment shall be selected in accordance with the preliminary plan preparation process discussed in 14.1.

2. Preliminary CIP wall design may assume a stem top width of 12 inches. Stem thickness at the bottom is based on load requirements and/or batter. The front batter of the stem should be set at ¼ inch per foot for stem heights up to 28 feet. For stem heights from 16 feet to 26 feet inclusive, the back face batter shall be a minimum of ½ inch per foot, and for stem heights of 28 ft maximum and greater, the back face shall be ¾ inch per foot per stability requirements.

3. Minimum Footing thickness for stem heights equal to or less than 10 ft shall be 1.5 ft and 2.0 ft when the stem height exceeds 10 ft or when piles are used.

4. The base of the footing shall be placed below the frost line, or 4 feet below the finished ground line. Selection of shallow footing or deep foundation shall be based on the geotechnical investigation, which should be performed in accordance with guidelines presented in Chapter 11 - Foundation Support.

5. The final footing embedment shall be based on wall stability requirements including bearing resistance, wall settlement limitations, external stability, internal stability and overall stability requirements.

6. If the finished ground line is on a grade, the bottom of footings may be sloped to a maximum grade of 12 percent. If the grade exceeds 12 percent, place the footings level and use steps.

The designer has the option to vary the values of each wall component discussed in steps 2 to 6 above, depending on site requirements and to achieve economy. See Figure 14.5-2 for initial wall sizing guidance.

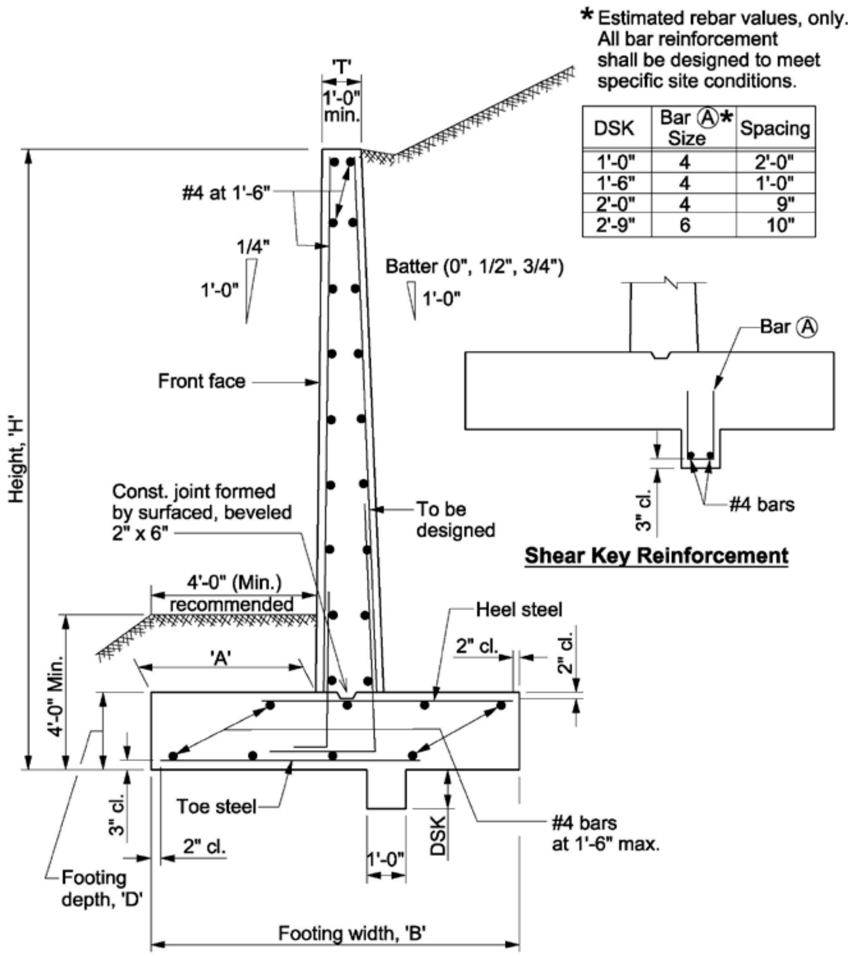


Figure 14.5-2
CIP Walls General Details

14.5.3.1 Wall Back and Front Slopes

CIP walls shall not be designed for backfill slope steeper than 2:1(H:V). Where practical, walls shall have a horizontal bench of 4.0 feet wide at the front face.

14.5.4 Unfactored and Factored Loads

Unfactored loads and moments are computed after establishing the initial wall geometry and using procedures defined in 14.4.5.4.5. A load diagram as shown in Figure14.4-1 for the earth pressure is developed assuming a triangular distribution plus additional pressures resulting from earth surcharge, water pressure, compaction or any other loads, etc. The material properties for backfill soil, concrete and steel are given in 14.4.6. The foundation and retained earth properties as recommended in the Geotechnical Report shall be used for computing nominal loads.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. Figure 14.4-8 shows load factor and load combinations along with their application for the load limit state evaluation. A summary of load factors and load combinations as applicable for a typical CIP cantilever wall is presented in Table 14.4-1 and **LRFD [3.4.1]**, respectively. Computed factored loads and moments are used for performing stability checks.

14.5.5 External Stability Checks

The external stability check includes checks for limiting eccentricity (overturning), bearing stress, and sliding at Strength I and Extreme Event II due to vehicle impact in cases where live load traffic is carried.

14.5.5.1 Eccentricity Check

The eccentricity of the retaining wall shall be evaluated in accordance with **LRFD [11.6.3.3]**. The location of the resultant force should be within 1/3 of base width of the foundation centroid (e<B/3) for foundations on soil, and within 0.45 of the base width of the foundation centroid (e<0.45B) for foundations on rock. If there is inadequate resistance to overturning (eccentricity value greater than limits given above), consideration should be given to either increasing the width of the wall base, or providing a deep foundation.

14.5.5.2 Bearing Resistance

The bearing resistance shall be evaluated at the strength limit state using factored loads and resistances. Bearing resistance of the walls founded directly on soil or rock shall be computed in accordance with 11.2 and **LRFD [10.6]**. The Bearing Resistance for walls on piles shall be computed in accordance with 11.3 and **LRFD [10.6]**. Figure 14.5-3 shows bearing stress criteria for a typical CIP wall on soil and rock respectively.

The vertical stress for footings on soil shall be calculated using:

$$\sigma_v = \frac{\sum V}{(B-2e)}$$

For walls founded on rock, the vertical stress is calculated assuming a linearly distributed pressure over an effective base area. The vertical stress for footings on rock shall be computed using:

$$\sigma_v = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right)$$

14.5.10 Summary of Design Requirements

1. Stability Check

   a. Strength I and Extreme Event II limit states

      - Eccentricity
      - Bearing Stress
      - Sliding

   b. Service I limit states

      - Overall Stability
      - Settlement

2. Foundation Design Parameters

   Use values provided by Geotechnical analysis

3. Concrete Design Data

   - f'c = 3500 psi
   - fy = 60,000 psi

4. Retained Soil

   - Unit weight = 120 lb/ft^3
   - Angle of internal friction - use value provided by Geotechnical analysis

5. Soil Pressure Theory

   - Coulomb theory for short heels or Rankine theory for long heels at the discretion of the designer.

6. Surcharge Load

   - Traffic live load surcharge = 2 feet = 240 lb/ft^2
   - If no traffic surcharge, use 100 lb/ft^2

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength I-a	0.90	1.00	1.75	1.75	1.50		Sliding, eccentricity
Strength I-b	1.25	1.35	1.75	1.75	1.50		Bearing /wall strength
Extreme II-a	0.90	1.00	-	-	-	1.00	Sliding, eccentricity
Extreme II-b	1.25	1.35	-	-	-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

Table 14.5-8
Load Factor Summary for CIP Walls

8. Bearing Resistance Factors

   - ϕ_b = 0.55 **LRFD [Table 11.5.7-1]**

9. Sliding Resistance Factors

   - ϕ_τ = 1.0 **LRFD [Table 11.5.7-1]**
   - ϕ_{ep} = 0.5 **LRFD Table [10.5.5.2.2-1]**

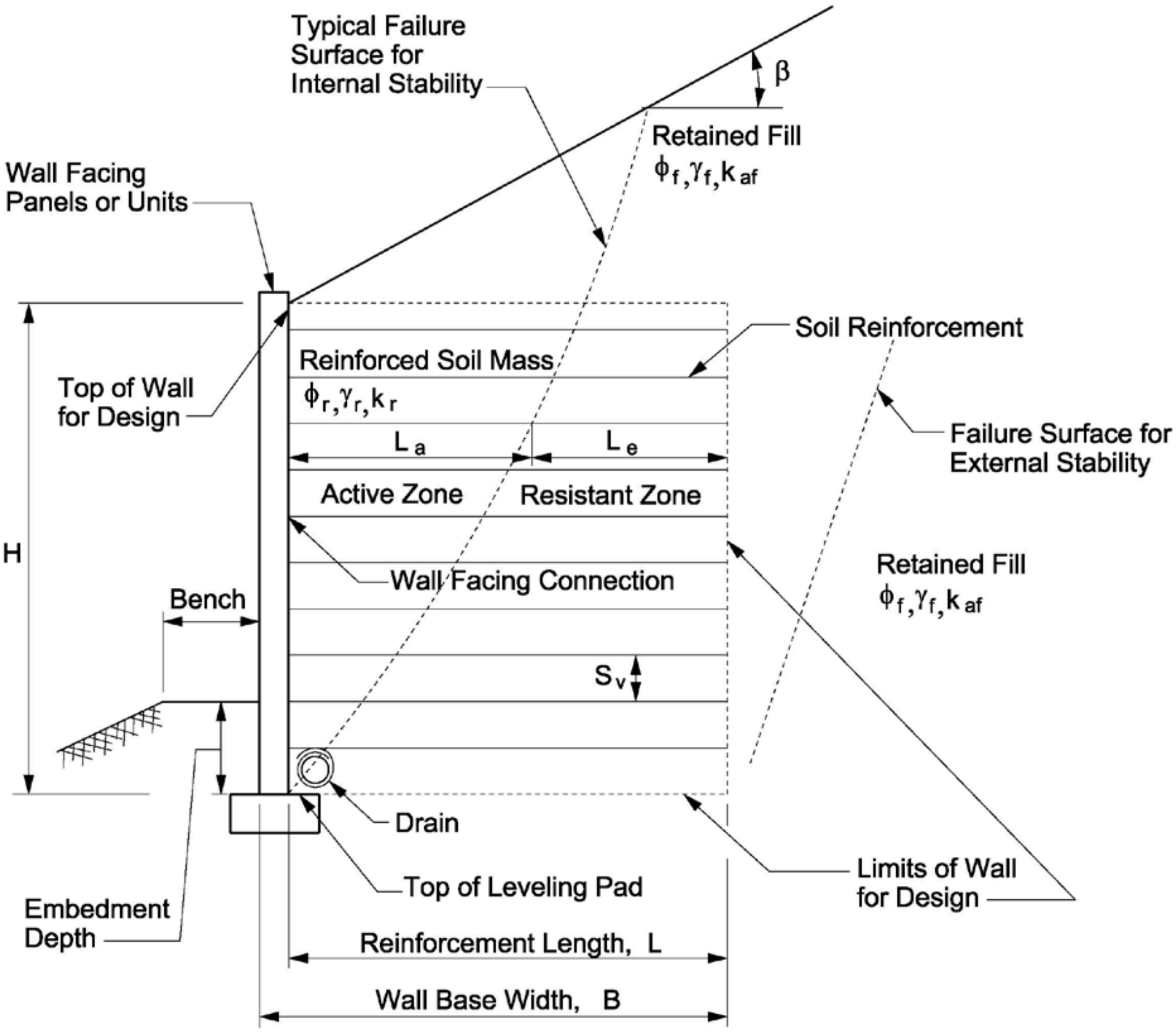


Figure 14.6-1
Structural Components of MSE Walls

14.6.2.1 Reinforced Earthfill Zone

The reinforced backfill to be used to construct the MSE wall shall meet the criteria in the wall specifications. The backfill shall be free from organics, or other deleterious material. It shall not contain foundry sand, bottom ash, blast furnace slag, or other potentially corrosive material. It shall meet the electrochemical criteria given in Table 14.6-1.

Reinforcement Material	Property	Criteria
Metallic	Resistivity	> 3000 ohm-cm
Metallic	Chlorides	< 100 ppm
Metallic	Sulfates	< 200 ppm
Metallic	pH	5.0 < pH < 10.0
Geosynthetic	pH	4.5 < pH < 9.0
Metallic/Geosynthetic	Organic Content	< 1.0 %

Table 14.6-1
Electrochemical Properties of Reinforced Fill MSE Walls

An angle of internal friction of 30 degrees and unit weight of 120 pcf shall be used for the stability analyses as stated in 14.4.6. If it is desired to use an angle of internal friction greater than 30 degrees, it shall be determined by the most current wall specifications.

14.6.2.2 Reinforcement:

Soil reinforcement can be either metallic (strips or bar grids like welded wire fabric) or non-metallic including geotextile and geogrids made from polyester, polypropylene, or high density polyethylene. Metallic reinforcements are also known as inextensible reinforcement and the non-metallic as extensible. Inextensible reinforcement deforms less than the compacted soil infill used in MSE walls, whereas extensible reinforcement deforms more than compacted soil infill

The metallic or inextensible reinforcement is mild steel, and usually galvanized or epoxy coated. Three types of steel reinforcement are typically used:

<u>Steel Strips</u>: The steel strip type reinforcement is mostly used with segmental concrete facings. Commercially available strips are ribbed top and bottom, 2 to 4 inch wide and 1/8 to 5/32 inch thick.

<u>Steel grids</u>: Welded wire steel grids using two to six W7.5 to W24 longitudinal wires spaced either at 6 or 8 inches. The transverse wire may vary from W11 to W20 and are spaced from 9 to 24 inches apart.

<u>Welded wire mesh</u>: Welded wire meshes spaced at 2 by 2 inch of thinner steel wire can also be used.

The galvanized steel reinforcement that is used for soil reinforcement is oversized in cross sectional areas to account for the corrosion that occurs during the life of the structure and the resulting loss of section. The net section remaining after corrosion at the end of the design service life is used to check design requirements

MSE Wire-Faced Facing

Welded wire fabric facing is used to build MSE wire-faced walls. These are essentially MSE walls with a welded wire fabric facing instead of a precast concrete facing. The wire size, spacing and patterns used in the facing are developed from performance data of full size wall tests and from applications in actual walls. A test to determine the connection strength between the soil reinforcement and the facing panels is required. Some systems do not use a connection because the ground reinforcement and facing panel are of one piece construction.

MSE wire-faced wall systems usually incorporate a backing mat behind the front facing. A fine metallic screen and geotextile fabric is placed behind the backing mat (or behind the facing if a backing mat is not used) to prevent the backfill from passing thru the front face.

MSE wire-faced walls can tolerate considerable differential settlement because of the flexibility of the wire facing. The limiting differential settlement is 1/50. The flexibility of the wire facing results in face bulging between ground reinforcement. The actual amount varies per system but normally is less than one inch. Recommended limits on bulging are 2" for permanent walls and 3" for temporary walls. This type of wall works well when a permanent wall facing can be placed after settlement/movement has occurred.

When MSE wire-faced walls are used for permanent wall applications, all steel components must be galvanized. When used for temporary wall applications black steel (non-galvanized) may be used since the walls are usually left in place and buried.

Temporary MSE wire-faced walls can be used as temporary shoring if site conditions permit. This wall type can also be used when staged construction is required to maintain traffic when an existing roadway is being raised and/or widened in conjunction with bridge approaches, railroad crossings or road reconstruction.

Cast-In- Place Concrete Facing

MSE walls with cast in place concrete facings are identical to MSE wire faced walls except a cast-in-place concrete facing is added after the wire face wall is erected. Modifications are made to the standard wire face wall detail to anchor the concrete facing to the wire facing and soil reinforcement. They are usually used when a special aesthetic facial treatment is required without the numerous joints that are common to precast panels. They can also be used where differential or total settlement is above tolerable limits for other wall types. A MSE wire faced wall can be constructed and allowed to settle with the concrete facing added after consolidation of the foundation soils has occurred.

The cast-in-place concrete facing shall be a minimum of 8-inches thick and contain coated or galvanized reinforcing steel. This is required because the panels and/or anchor that extend into the cast-in-place concrete are galvanized and a corrosion cell would be created if black steel contacts galvanized steel. All wire ties and bar chairs used in the cast–in-place concrete must also be coated or galvanized. Note that the 8-inch minimum wall thickness will occur at the points of maximum panel bulging and that the wall will be thicker at other locations. Also note that the 8-inch minimum is measured from the trough of any form liner or rustication.

Vertical construction joints are required in the cast-in-place concrete facing to allow for expansion and contraction and to allow for some differential settlement. Closer spacing of vertical construction joints is required when differential settlement may occur, but by delaying the placement of the cast-in-place concrete, the effects of differential settlement is minimized. Higher walls also require closer spacing of vertical construction joints if differential settlement is anticipated. Horizontal construction joints may disrupt the flow of a special aesthetic facial treatment and are sometimes not allowed for that reason. The designer should specify if optional horizontal construction joints are allowed. Cork filler is placed at vertical construction joints because cork is compressible and will allow some expansion and rotation to occur at the joint. An expandable polyvinyl chloride waterstop (PCW) is used on the back side of a vertical construction joint. Since forms are only used at the front face of the wall the PCW can be attached to a 10-inch board which is supported by the wire facing. The 8-inch minimum wall thickness may be decreased at the location of the vertical construction joint to accommodate the PCW and its support board.

Geosynthetic Facing

Geosynthetic reinforcements are looped around at the facing to form the exposed face of the MSE Wall. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire. Geogrid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. This facing is generally used in temporary applications. Similar to wire faced walls, these walls typically have a geotextile behind the geogrids, to prevent material from passing through the face.

14.6.3 Design Procedure

14.6.3.1 General Design Requirements

The procedure for design of an MSE wall requires evaluation of external stability and internal stability (structural design) at Strength Limit States and overall stability and vertical/lateral movement at Service Limit State. The Extreme Event II load combination is used to design and analyze for vehicle impact where traffic barriers are provided to protect MSE walls. The design and stability is performed in accordance with *AASHTO LRFD* and design guidance discussed in 14.4.

14.6.3.2 Design Responsibilities

MSE walls are proprietary wall systems and the structural design of the wall system is provided by the contractor. The structural design of the MSE wall system must include an analysis of internal stability (soil reinforcement pullout and stress) and local stability (facing connection forces and internal panel stresses). Additionally, the contractor should also provide internal drainage. Design drawings and calculations must be submitted to the Bureau of Structures for acceptance.

External stability, overall stability and settlement calculations are the responsibility of the WISDOT/Consultant designer. Compound stability is the responsibility of the Contractor. Soil borings and soil design parameters are provided by Geotechnical Engineer.

Although abutment loads can be supported on spread footings within the reinforced soil zone, it is WisDOT policy to support the abutment loads for multiple span structures on piles or shafts that pass through the reinforced soil zone to the in-situ soil below. Piles shall be driven prior to the placement of the reinforced earth. Strip type reinforcement can be skewed around the piles but must be connected to the wall panels and must extend to the rear of the reinforced soil zone.

For continuous welded wire fabric reinforcement, the contractor should provide details on the plans showing how to place the reinforcement around piles or any other obstacle. Abutments for single span structures may be supported by spread footings placed within the soil reinforcing zone, with WISDOT's approval. Loads from such footings must be considered for both internal wall design and external stability considerations.

14.6.3.3 Design Steps

Design steps specific to MSE walls are described in FHWA publication No*. FHWA-NHI-10-24* and modified shown below:

1. Establish project requirements including all geometry, loading conditions (transient and/or permanent), performance criteria, and construction constraints.
2. Evaluate existing topography, site subsurface conditions, in-situ soil/rock properties, and wall backfill parameters.
3. Select MSE wall using project requirement per step 1 and wall selection criteria discussed in 14.3.1.
4. Based on initial wall geometry, estimate wall embedment depth and length of reinforcement.
5. Estimate unfactored loads including earth pressure for traffic surcharge or sloping back slope and /or front slope.
6. Summarize load factors, load combinations, and resistance factors
7. Calculate factored loads for all appropriate limit states and evaluate (external stability) at Strength I Limit State
    a. sliding
    b. eccentricity
    c. bearing
8. Compute settlement at Service limit states
9. Compute overall stability at Service limit states
10. Compute vertical and lateral movement
11. Design wall surface drainage systems
12. Compute internal stability
    a. Select reinforcement
    b. Estimate critical failure surface
    c. Define unfactored loads
    d. Calculate factored horizontal stress and maximum tension at each reinforcement level
    e. Calculate factored tensile stress in each reinforcement
    f. Check factored reinforcement pullout resistance
    g. Check connection resistance requirements at facing
13. Design facing element
14. Design subsurface drainage

Steps 1-11 are completed by the designer and steps 12-14 are completed by the contractor after letting.

14.6.3.4 Initial Geometry

Figure 14.6-1 provides MSE wall elements and dimensions that should be established before making stability computations for the design of an MSE wall. The height (H) of an MSE wall is measured vertically from the top of the MSE wall to the top of the leveling pad. The length of reinforcement (L) is measured from the back of MSE wall panels. Alternately, the length of reinforcement (L1) is measured from the front face for modular block type MSE walls.

The MSE walls, with panel type facings, generally do not exceed heights of 35 feet, and with modular block type facings, should not exceed heights of 22 feet. Wall heights in excess of these limits will require approval on a case by case basis from WisDOT.

In general, a minimum reinforcement length of 0.7H or 8 feet whichever is greater shall be provided. MSE wall structures with sloping surcharge fills or other concentrated loads will generally require longer reinforcement lengths of 0.8H to 1.1H. As an exception, a minimum reinforcement length of 6.0 feet or 0.7H may be provided in accordance with **LRFD [C11.10.6.2.1]** provided all conditions for external and internal stability are met and smaller compaction equipment is used on a case by case basis as approved by WisDOT. MSE walls may be built to heights mentioned above; however, the external stability requirements may limit MSE wall height due to bearing capacity, settlement, or stability problems.

14.6.3.4.1 Wall Embedment

The minimum wall embedment depth to the bottom of the MSE wall reinforced backfill zone (top of the leveling pad shown in **LRFD [Figure 11.10.2-1]** and Figure 14.6-1 shall be based on external stability analysis (sliding, bearing resistance, overturning, and settlement) and the global (overall) stability requirements.

Minimum MSE wall leveling pad (and front face) embedment depths below lowest adjacent grade in front of the wall shall be in accordance with **LRFD [11.10.2.2]**, including the minimum embedment depths indicated in **LRFD [Table C11.10.2.2-1]** or 1.5 ft. whichever is greater. The embedment depth of MSE walls along streams and rivers shall be at least 2.0 ft below the potential scour elevation in accordance with **LRFD [11.10.2.2]** and the *Bridge Manual*.

WisDOT policy item:

The minimum depth of embedment of MSE walls shall be 1.5 feet

14.6.3.4.2 Wall Backslopes and Foreslopes

The wall Backslopes and Foreslopes shall be designed in accordance with 14.4.5.4.4. A minimum horizontal bench width of 4 ft (measured from bottom of wall horizontally to the slope face) shall be provided, whenever possible, in front of walls founded on slopes. This minimum bench width is required to protect against local instability near the toe of the wall.

14.6.3.5 External Stability

The external stability of the MSE walls shall be evaluated for sliding, limiting eccentricity, and bearing resistance at the Strength I limit state. The settlement shall be calculated at Service I limit state.

Unfactored loads and factored load shall be developed in accordance with 14.6.3.5.1. It is assumed that the reinforced mass zone acts as a rigid body and that wall facing, the reinforced soil and reinforcement act as a rigid body.

For adequate stability, the goal is to have the factored resistance greater than the factored loads. According to publication FHWA-NHI-10-024, a capacity to demand ratio (CDR) can be used to quantify the factored resistance and factored load. CDR has been used to express the safety of the wall against sliding, limiting eccentricity, and bearing resistance.

14.6.3.5.1 Unfactored and Factored Loads

Unfactored loads and moments are computed based on initial wall geometry and using procedures defined in 14.4.5.4.5. The loading diagrams for one of the 3 possible earth pressure conditions are developed. These include 1) horizontal backslope with traffic surcharge shown in Figure 14.4-2; 2) sloping backslope shown in Figure 14.4-3; and, 3) broken backslope condition as shown in Figure 14.4-4.

The computed nominal loads discussed in 14.5.4 are multiplied by applicable load factors given in Table 14.4-1. A summary of load factors and load combinations as applicable for typical MSE wall stability check is presented in Table 14.6-4. Computed factored load and moments are used for performing stability checks.

14.6.3.5.2 Sliding Stability

The stability should be computed in accordance with **LRFD [11.10.5.3]** and **LRFD [10.6.3.4]**. The sliding stability analysis shall also determine the minimum resistance along the following potential surfaces in the zones shown in **LRFD [Figure 11.10.2.1]**.

- Sliding within the reinforced backfill (performed by contractor)
- Sliding along the reinforced back-fill/base soil interface (performed by designer)

The coefficient of friction angle shall be determined as:

- For discontinuous reinforcements, such as strips – the lesser of friction angle of either reinforced backfill, ϕ_r, the foundation soil, ϕ_{fd}.
- For continuous reinforcements, such as grids and sheets – the lesser of ϕ_r or ϕ_{fd} and ρ.

No passive soil pressure is allowed to resist sliding. The component of the passive resistance shall be ignored due to the possibility that permanent or temporary excavations in front of the wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance. The shear strength of the facing system is also ignored.

For adequate stability, the factored resistance should be greater than the factored load for sliding,

The following equation shall be used for computing sliding:

$R_\tau = \phi\ R_n = \phi_\tau$*(V)*(tan$\delta$)

Where:

R_R	=	Factored resistance against failure by sliding
R_n	=	Nominal sliding resistance against failure by sliding
R_τ	=	Nominal sliding resistance between soil and foundation
ϕ_τ	=	Resistance factor for shear between the soil and foundation per **LRFD [Table 11.5.6.1]**; 1.0
V	=	Factored vertical dead load
δ	=	Friction angle between foundation and soil
ρ	=	Maximum soil reinforcement interface angle **LRFD [11.11.5.3]**
tanδ	=	tan ϕ_{fd} where ϕ is lesser of (ϕ_τ , ϕ_{fd} , ρ)
H_{tot}	=	Factored total horizontal load for Strength Ia
CDR	=	$R_\tau/H_{tot} \geq 1$

14.6.3.5.3 Eccentricity Check

The eccentricity check is performed in accordance with **LRFD [11.6.3.3]** and using procedure given in publication, *FHWA-NHI-10-025*

The eccentricity is computed using:

$e = B/2 - X_0$

Where:

$$X_0 = \frac{\sum M_V - M_H}{\Sigma V}$$

Where:

ΣM_V = Summation of Resisting moment due to vertical earth pressure

ΣM_H = Summation of Moments due to Horizontal Loads

ΣV = Summation of Vertical Loads

For eccentricity to be considered acceptable, the calculated location of the resultant vertical force (based on factored loads) should be within the middle two-thirds of the base width for soil foundations (i.e., e_{max} = B/3) and middle nine-tenths of the base width for rock foundations (i.e., e_{max} = 0.45B). Therefore, for each load group, e must be less than e_{max}. If e is greater than e_{max}, a longer length of reinforcement is required. The CDR for eccentricity should be greater than 1.

$CDR = e_{max}/e > 1$

14.6.3.5.4 Bearing Resistance

The bearing resistance check shall be performed in accordance with **LRFD [11.10.5.4]**. Provisions of **LRFD [10.6.3.1]** and **LRFD [10.6.3.2]** shall apply**.** Because of the flexibility of MSE walls, an equivalent uniform base pressure shall be assumed. Effect of live load surcharge shall be added, where applicable, because it increases the load on the foundation. Vertical stress, $\sigma_{v,}$ shall be computed using following equation.

The bearing resistance computation requires:

Base Pressure $(\sigma_v) = \dfrac{\sum V}{B-2e}$

σ_v = Vertical pressure

ΣV = Sum of all vertical forces

B = Reinforcement length

e = Eccentricity = $B/2 - X_0$

X_0 = $(\Sigma M_R - \Sigma M_H)/\Sigma V$

ΣM_V = Total resisting moments

ΣM_H = Total driving moments

The nominal bearing resistance, q_n, shall be computed using methods for spread footings. The appropriate value for the resistance factor shall be selected from **LRFD [Table 11.5.7-1]**.

The computed vertical stress, σ_v, shall be compared with factored bearing resistance, q_r in accordance with the **LRFD [11.10.5.4]** and a Capacity Demand Ratio, CDR, shall be calculated using the following equation:

$$q_r = \phi_b \, q_n \geq \sigma_v$$

Where:

q_r	=	Factored bearing resistance
q_n	=	Nominal bearing resistance computed using **LRFD [10.6.3.1.2a-1]**
ϕ_b	=	0.65 using **LRFD [Table 11.5.7-1]**
CDR	=	q_r/σ_v >1.0

14.6.3.6 Vertical and Lateral Movement

Excessive MSE wall foundation settlement can result in damage to the wall facing, coping, traffic barrier, bridge superstructure, bridge end panel, pavement, and/or other settlement-sensitive elements supported on or near the wall.

Techniques to reduce damage from post-construction settlements and deformations may include full-height vertical sliding joints through the rigid wall facing elements and appurtenances, and/or ground improvement or reinforcement techniques. Staged preload/surcharge construction using onsite materials or imported fills may also be used.

Settlement shall be computed using the procedures outlined in 14.4.7.2 and the allowable limit settlement guidelines in 14.4.7.2.1 and in accordance with **LRFD [11.10.4]** and **LRFD [10.6.2.4].** Differential settlement from the front face to the back of the wall shall be evaluated, as appropriate.

For MSE walls with rigid facing concrete panels, slip joints of 0.75 inch width can be provided to control differential settlement as per **LRFD [Table C11.104.4-1]**.

14.6.3.7 Overall Stability

Overall Stability shall be performed in accordance with **LRFD [11.10.4.3]**. Provision of **LRFD [11.6.2.3]** shall also apply. Overall and compound stability of complex MSE wall system shall also be investigated, especially where the wall is located on sloping or soft ground where overall stability may be inadequate. Compound external stability is the responsibility of the contractor/wall supplier. The long term strength of each backfill reinforcement layer intersected by the failure surface should be considered as restoring forces in the limit equilibrium slope stability analysis. Figure 14.6-4 shows failure surfaces generated during overall or compound stability evaluation.

T_{al} = Nominal tensile resistance (reinforcement design strength) at each reinforcement level

The value for T_{MAX} is calculated with a load factor of 1.35 for vertical earth pressure, EV. The tensile resistance factor for metallic and geosynthetic reinforcement is based on the following:

Metallic Reinforcement	Strip Reinforcement • Static Loading Grid Reinforcement • Static Loading	 0.75 0.65
Geosynthetic reinforcement	• Static Loading	0.90

Table 14.6-3
Resistance Factor for Tensile and Pullout Resistance
(Source **LRFD [Table 11.5.7.1]**)

14.6.3.8.7 Calculate T_{al} for Inextensible Reinforcements

T_{al} for inextensible reinforcements is computed as below:

$T_{al} = (A_c\ F_y)/b$

Where:

F_y = Minimum yield strength of steel

b = Unit width of sheet grid, bar, or mat

A_c = Design cross sectional area corrected for corrosion loss

14.6.3.8.8 Calculate T_{al} for Extensible Reinforcements

The available long-term strength, T_{al}, for extensible reinforcements is computed as:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{Tult}{RF_{ID} * RF_{CR} * RF_D}$$

Where:

T_{ult} = Minimum average roll value ultimate tensile strength

RF = Combined strength reduction factor to account for potential long term degradation due to installation, damage, creep, and chemical aging

RF_{ID} = Strength Reduction Factor related to installation damage

RF_{CR} = Strength Reduction Factor caused by creep due to long term tensile load

RF_D = Strength Reduction Factor due to chemical and biological degradation

RF shall be determined from product specific results as specified in **LRFD [11.10.6.4.3b]**.

14.6.3.8.9 Design Life of Reinforcements

Long term durability of the steel and geosynthetic reinforcement shall be considered in MSE wall design to ensure suitable performance throughout the design life of the structure.

The steel reinforcement shall be designed to achieve a minimum designed life in accordance with **LRFD [11.5.1]** and shall follow the provision of **LRFD [7.6.4.2]**. The provision for corrosion loss shall be considered in accordance with the guidance presented in **LRFD [11.10.6.4.2a]**.

The durability of polymeric reinforcement is influenced by time, temperature, mechanical damage, stress levels, and changes in molecular structure. The strength reduction for geosynthetic reinforcement shall be considered in accordance with **LRFD [11.10.6.4.2b]**.

14.6.3.8.10 Reinforcement /Facing Connection Design Strength

Connections shall be designed to resist stresses resulting from active forces as well as from differential movement between the reinforced backfill and the wall facing elements in accordance with **LRFD [11.10.6.4.4]**.

Steel Reinforcement

Capacity of the connection shall be tested per **LRFD [5.11.3].** Elements of the connection which are embedded in facing elements shall be designed with adequate bond length and bearing area in the concrete, to resist the connection forces. The steel reinforcement connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4a]**.

Connections between steel reinforcement and the wall facing units (e.g. bolts and pins) shall be designed in accordance with **LRFD [6.1.3]**. Connection material shall also be designed to accommodate loss due to corrosion.

Geosynthetic Reinforcement

The portion of the connection embedded in the concrete facing shall be designed in accordance with **LRFD [5.11.3]**. The nominal geosynthetic connection strength requirement shall be designed in accordance with **LRFD [11.10.6.4.4b]**.

14.6.3.8.11 Design of Facing Elements

Precast Concrete Panel facing elements are designed to resist the horizontal forces developed internally within the wall. Reinforcement is provided to resist the average loading conditions at each depth in accordance with structural design requirements in *AASHTO LRFD*. The embedment of the reinforcement to panel connector must be developed by test, to ensure that it can resist the maximum tension. The concrete panel must meet temperature and shrinkage steel requirements. Epoxy protection of panel reinforcement is required.

Modular Block Facing elements must be designed to have sufficient inter-unit shear capacity. The maximum spacing between unit reinforcement should be limited to twice the front block width or 2.7 feet, whichever is less. The maximum depth of facing below the bottom reinforcement layer should be limited to the block width of modular facing unit. The top row of reinforcement should be limited to 1.5 times the block width. The factored inter-unit shear capacity as obtained by testing at the appropriate normal load should exceed the factored horizontal earth pressure.

14.6.3.8.12 Corrosion

Corrosion protection is required for all permanent and temporary walls in aggressive environments as defined in **LRFD [11.10.2.3.3].** Aggressive environments in Wisconsin are typically associated with salt spray and areas near storm water pipes in urban areas. MSE walls with steel reinforcement should be protected with a properly designed impervious membrane layer below the pavement and above the first level of the backfill reinforcement. The details of the impervious layer drainage collector pipe can be found in *FHWA-NHI-0043* (FHWA 2001).

14.6.3.9 Wall Internal Drainage

The wall internal drainage should be designed using the guidelines provided in 14.4.7.6. Pipe underdrain must be provided to properly drain MSE walls. Chimney or blanket drains with collector-pipe drains are installed as part of the MSE walls sub-drainage system. Collector pipes with solid pipes are required to carry the discharge away from the wall. All collector pipes and solid pipes should be 6-inch diameter.

14.6.3.10 Traffic Barrier

Design concrete traffic barriers on MSE walls to distribute applied traffic loads in accordance with **LRFD [11.10.10.2]** and WisDOT standard details. Traffic impact loads shall not be transmitted to the MSE wall facing. Additionally, MSE walls shall be isolated from the traffic barrier load. Traffic barrier shall be self-supporting and not rely on the wall facing.

14.6.3.11 Design Example

Example E-2 shows a segmental precast panel MSE wall with steel reinforcement. Example E-3 shows a segmental precast panel MSE wall with geogrid reinforcement. Both design examples include external and internal stability of the walls. The design examples are included in 14.18.

14.6.3.12 Summary of Design Requirements

1. Strength Limit Checks

   a. External Stability

   - Sliding

$$CDR = \left(\frac{R_\tau}{H_{tot}}\right) > 1.0$$

   - Eccentricity Check

$$CDR = \left(\frac{e_{\max}}{e}\right) > 1.0$$

   - Bearing Resistance

$$CDR = \left(\frac{q_r}{\sigma_v}\right) > 1.0$$

   b. Internal stability

   - Tensile Resistance of Reinforcement
   - Pullout Resistance of Reinforcement
   - Structural resistance of face elements and face elements connections

   c. Service Limit Checks

   - Overall Stability
   - Wall Settlement and Lateral Deformation

2. Concrete Panel Facings

   - f'_c = 4000 psi (wet cast concrete)
   - Min. thickness = 5.5 inches
   - Min. reinforcement = 1/8 square inch per foot in each direction (uncoated)

front of the wall could occur during the service life of the structure and lead to partial or complete loss of passive resistance.

Interface sliding resistance between concrete blocks shall be calculated using the corrected wall weight based on the calculated hinge height in accordance with **LRFD Figure [11.10.6.4.4b-1]**. Interface friction resistance parameters shall be based on NCMA method. Shear between the blocks must be resisted by friction, keys or pins.

14.7.1.2.3 Bearing Resistance

The bearing resistance of the walls shall be computed in accordance with **LRFD [10.6.3.1]**.

Base Pressure, $$\sigma_v = \frac{\sum V_{tot}}{(B-2e)}$$

The computed vertical stress shall be compared with factored bearing resistance in accordance with the **LRFD [10.6.3.1]**, using following equation:

$$q_r = \phi_b q_n \geq \sigma_v$$

Where:

q_n	=	Nominal bearing resistance computed using **LRFD [10.6.3.1.2-a]**
$\sum V$	=	Summation of Vertical loads
B	=	Base width
e	=	Eccentricity
ϕ_b	=	0.55 **LRFD [Table 11.5.7-1]**

14.7.1.2.4 Eccentricity Check

The eccentricity check shall be performed in accordance with **LRFD [11.6.3.3].** The location of the resultant force should be within the middle two-thirds of the base width (e<B/3) for footings on soil, and within nine-tenths of the base (e<0.45B) for footings on rock.

14.7.1.3 Settlement

The vertical and lateral displacements of prefabricated modular retaining walls must be evaluated for all applicable dead and live load combinations at Service I limit states using procedures described in14.4.7.2 and compared with tolerable movement criteria presented in 14.4.7.2.1. In general, lateral movements of walls on shallow foundations can be estimated assuming the wall rotates or translates as a rigid body due to the effects of earth loads and differential settlements along the base of the wall.

14.7.1.4 Overall Stability

The overall (global) stability shall be evaluated in accordance with **LRFD [11.6.2.3]** and in accordance with 14.4.7.3, with the exception that the entire mass of the modular walls (or the “foundation load”), may be assumed to contribute to the overall stability of the slope. The overall stability check shall be performed by the Geotechnical Engineering Unit or Consultant of record.

14.7.1.5 Summary of Design Requirements

1. Stability Evaluations
   - External Stability
     - Eccentricity Check
     - Bearing Check
     - Sliding
   - Settlement
   - Overall/Global
2. Block Data
   - One piece block
   - Minimum thickness of front face = 4 inches
   - Minimum thickness of internal cavity walls other than front face = 2 inches
   - 28 day concrete strength = 5000 psi
   - Maximum water absorption rate by weight = 5%
3. Traffic Surcharge
   - Traffic live load surcharge = 240 lb/ft^2
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment
4. Retained Soil
   - Unit weight γ_f = 120 lb/ft^3
   - Angle of internal friction as determined by Geotechnical Engineer

5. Soil Pressure Theory

   - Use Coulomb Theory

6. Maximum Height = 8 ft.

   (This height is measured from top of leveling pad to bottom of cap. It is not the exposed height). In addition this maximum height may be reduced if there is sloping backfill or a sloping surface in front of the wall.)

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength Ia	0.90	1.00	0.0	1.75	1.50		Sliding, eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50		Bearing /wall strength
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

Table 14.7-1
Load Factor Summary for Prefabricated Modular Walls

8. Sliding Resistance Factors

   ϕ_{τ} = 1.0 **LRFD [Table11.5.7-1]**

9. Bearing Resistance Factors

   ϕ_b = 0.55 **LRFD [Table11.5.7-1]**

14.8 Prefabricated Modular Walls

Prefabricated modular walls systems use interconnected structural elements, which use selected in-fill soil or rock fill to resist external pressures by acting as gravity retaining walls. Metal and precast concrete or metal bin walls, crib walls, and gabion walls are considered under the category of prefabricated modular walls. These walls consist of modular elements which are proprietary. The design of these wall systems is provided by the contractor/wall supplier.

Prefabricated modular walls can be used where reinforced concrete walls are considered. Steel modular systems should not be used where aggressive environmental condition including the use of deicing salts or other similar chemicals are used that may corrode steel members and shorten the life of modular wall systems.

14.8.1 Metal and Precast Bin Walls

Metal bin walls generally consist of sturdy, lightweight, modular steel members called as stringers and spacers. The stringers constitute the front and back face of the bin and spacers its sides. The wall is erected by bolting the steel members together. The flexibility of the steel structure allows the wall to flex against minor ground movement. Metal bin walls are subject to corrosion damage from exposure to water, seepage and deicing salts. To improve the service life of metal bin walls, consideration should be given towards increasing the galvanizing requirements and establishing electrochemical requirements for the confined backfill.

Precast concrete bin walls are typically rectangular interlocking prefabricated concrete modules. A common concrete module typically has a face height varying from 4 to 5 feet, a face length up to 8 feet, and a width ranging from 4 to 20 feet. The wall can be assembled vertically or provided with a batter. A variety of surface treatment can be provided to meet aesthetic requirements. A parapet wall can be provided at the top of the wall and held rigidly by a cast in place concrete slab. A reinforced cast-in-place or precast concrete footing is usually placed at the toe and heel of the wall.

Bin walls are not recommended for applications that require a radius of curvature less than 800 ft. The wall face batter shall not be steeper than 10° or 6:1 (V:H). The base width of bin walls is generally 60% of the wall height. Further description and method of construction can be found in FHWA's publication *Earth Retaining Structures* 2008.

14.8.2 Crib Walls

Crib walls are built using prefabricated units which are stacked and interlocked and filled with free draining material. Cribs consist of solid interlocking reinforced concrete members called rails and tiebacks (sometimes called stretchers and headers). The rails run parallel with the wall face at both the front and rear of the cribbing and the tiebacks run transverse to the rails to tie the structure together. Rails and cross sections of tiebacks form the front face of the wall.

The wall face can either be opened or closed. In closed faced cribs, stretchers are placed in contact with each other. In open face cribs, the stretchers are placed at an interval such that the infill material does not escape through the face. The wall face batter for crib walls shall be no steeper than 4:1.

14.8.3 Gabion Walls

The gabion walls are composed of orthogonal wire cages or baskets tied together and filled with rock fragments. These wire baskets are also known as gabion baskets. The basket size can be varied to suit the terrain with a standard width of 3 feet to standard length varying 3 to 12 feet. The standard height of these baskets may vary from 1 foot to 3 feet. Individual wire baskets are filled with rock fragments ranging in size from 4 to 10 inches. After the baskets are filled, the lids are closed and wired shut to form a relatively rigid block. Succeeding rows of the gabions are laced in the field to the underlying gabions and are filled in the same manner until the wall reaches its design height. The rock filled baskets are closed with lids.

The durability of a gabion wall is dependent upon maintaining the integrity of the gabion baskets. Galvanized steel wire is required for all gabion installations. Although gabions are manufactured from a heavy gage wire, there is a potential for damage due to vandalism. While no known case of such vandalism has occurred on any existing WisDOT gabion walls, the potential for such action should be considered at specific sites.

A height of about 18 feet should be considered as a practical limit for gabion walls. Gabion walls have shown good economy for low to moderate heights but lose this economy as height increases. The front and rear face of the wall may be vertical or stepped. A batter is provided for walls exceeding heights of 10 feet, to improve stability. The wall face step shall not be steeper than 6” or 10:1(V:H). The minimum embedment for gabion walls is 1.5 feet. The ratio of the base width to height will normally range from 0.5 to 0.75 depending on backslope, surcharge and angle of internal friction of retained soil. Gabion walls should be designed in cross section with a horizontal base and a setback of 4 to 6 inches at each basket layer. This setback is an aid to construction and presents a more pleasing appearance. The use of a tipped wall base should not be allowed except in special circumstances.

14.8.4 Design Procedure

All prefabricated modular wall systems shall be designed to resist external pressure caused by the supported earth, surcharge loads, and water in accordance with design criteria discussed in **LRFD [11.11.4]** and 14.4 of this chapter. The design requires an external stability evaluation by the WISDOT/Consultant designer, including sliding, eccentricity, and bearing resistance check at the Strength I limit state and the evaluation of wall settlement and overall stability at the Service I limit state.

In addition, the structures modules of the bin and crib walls shall be designed to provide adequate resistance against structural failure as part of the internal stability evaluations in accordance with the guidelines presented in **LRFD [11.11.5]**.

No separate guidance is provided in the *AASHTO LRFD* for the gabion walls, therefore, gabion walls shall be evaluated for the external stability at Strength I and the settlement and overall stability checks at Service I using similar process as that of a prefabricated modular walls.

Since structure modules of the prefabricated modular walls are proprietary, the contractor/ supplier is responsible for the internal stability evaluation and the structural design of the structural modules. The design by contractor shall also meet the requirements for any special provisions. The external stability, overall stability check and the settlement evaluation will be performed by Geotechnical Engineer.

14.8.4.1 Initial Sizing and Wall Embedment

Wall backfill shall not be steeper than 2:1(V:H). Where practical, a minimum 4.0 feet wide horizontal bench shall be provided in front of the walls. A base width of 0.4 to 0.5 of the wall height can be considered initially for walls with no surcharge. For walls with surcharge loads or larger backslopes, an initial base width of 0.6 to 0.7 times can be considered.

Wall embedment for prefabricated modular walls shall meet the requirements discussed in 14.4.7.5. A minimum embedment shall be 1.5 ft or the requirement for scouring or erosion due to flooding.

14.8.5 Stability checks

Stability computations for crib, bin, and gabion modular wall systems shall be made by assuming that the wall modules and wall acts as a rigid body. Stability of gabion walls shall be performed assuming that gabions are flexible.

14.8.5.1 Unfactored and Factored Loads

All modular walls shall be investigated for lateral earth and water pressure including any live and/or dead load surcharge. Dead load due to self weight and soil or rock in-fill shall also be included in computing the unfactored loads. Material properties for selected backfill, concrete, and steel shall be in accordance with guidelines suggested in 14.4.6. The properties of prefabricated modules shall be based on the type of wall modules being supplied by the wall suppliers.

The angle of friction δ between the back of the modules and backfill shall be used in accordance with the **LRFD [3.11.5.9]** and **LRFD [Table C3.11.5.9.1]**. Loading and earth pressure distribution diagram shall be developed as shown in Figure 14.4-6 or Figure 14.4-7

Since infill material and backfill materials of the gabion walls are well drained, no hydrostatic pressure is considered for the gabion walls. The unit weight of the rock-filled gabion baskets shall be computed in accordance with following:

$$\gamma_g = (1-\eta_r)G_s\gamma_w$$

Where:

η_r = Porosity of the rock fill

Gs = Specific gravity of the rock

- 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, subdrainage installation, reinforcement, and temporary shotcrete placement. Soil nail walls should not be used below groundwater.

14.9.1 Design Requirements

AASHTO LRFD currently does not include the design and construction of soil nail walls. It is recommended that soil nail walls be designed using methods recommended in *Geotechnical Engineering Circular (GEC) No. 7 – Soil Nail Walls* (FHWA, 2003). The design life of the soil nail walls shall be in accordance with 14.4.3.

The design of the soil nailing walls requires an evaluation of external, internal, and overall stability and facing-connection failure mode as presented in Sections 5.1 thru Sections 5.6 of *(GEC) No. 7 – Soil Nail Walls* (FHWA, 2003).

A permanent wall facing is required for all permanent soil nail walls. Permanent facing is commonly constructed of cast-in-place (CIP) concrete, welded wire mesh (WWM) reinforced concrete and precast fabricated panels. In addition to meeting the aesthetic requirements and providing adequate corrosion protections to the soil nails, design facings for all facing-connection failure modes indicated in FHWA 2003.

Corrosion protection is required for all permanent soil nail wall systems to assure adequate long-tem wall durability. . The level of corrosion protection required should be determined on a project-specific basis based on factors such as wall design life, structure criticality and the electrochemical properties of the supporting soil and rock materials. Criteria for classification of the supporting soil and rock materials as "aggressive" or "non-aggressive" are provided in FHWA 2003.

Soil nails are field tested to verify that nail design loads can be supported without excessive movement and with an adequate margin of safety. Perform both verification and proof testing of designated test nails as recommended in FHWA 2003.

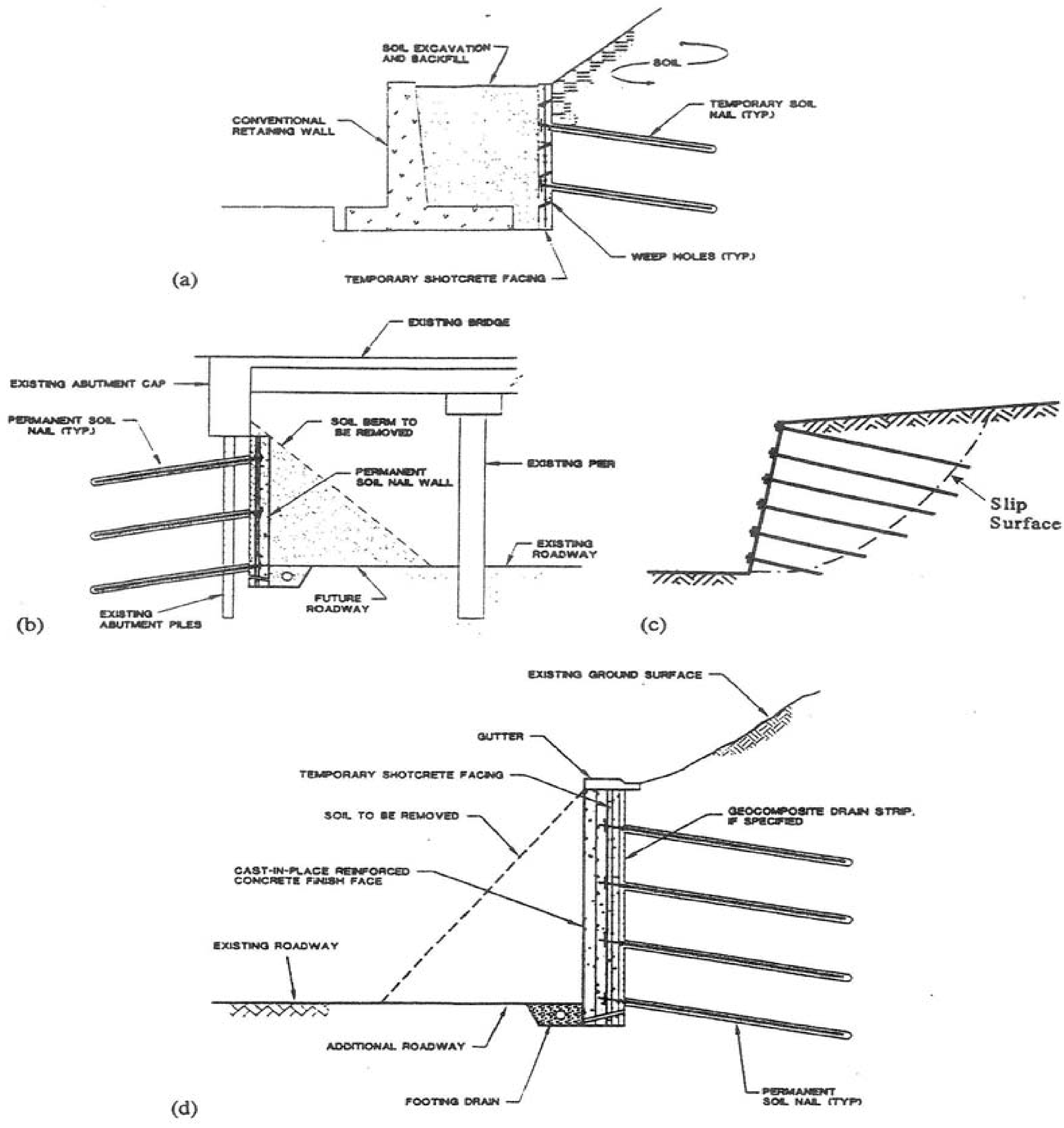


Figure 14.9-1
In-Situ Soil Nailed Walls
(Source: Earth Retaining Structures, 2008)

14.10 Steel Sheet Pile Walls

14.10.1 General

Steel sheet pile walls are a type of non-gravity wall and are typically used as temporary walls, but can also be used for permanent locations.

Sheet piling consists of interlocking steel, precast concrete or wood pile sections driven side by side to form a continuous unit. Steel is used almost exclusively for sheet pile walls. Individual pile sections usually vary from 12 to 21 inches in width, allowing for flexibility and ease of installation. The most common use of sheet piling is for temporary construction of cofferdams, retaining walls or trench shoring. The structural function of sheet piles is to resist lateral pressures due to earth and/or water. The steel manufacturers have excellent design references. Sheet pile walls generally derive their stability from sufficient pile penetration (cantilever walls). When sheet pile walls reach heights in excess of approximately 15 feet, the lateral forces are such that the walls need to be anchored with some form of tieback.

Cofferdams depend on pile penetration, ring action and the tensile strength of the interlocking piles for stability. If a sheet pile cofferdam is to be dewatered, the sheets must extend to a sufficient depth into firm material to prevent a "blow out", that is water coming in from below the base of the excavation. Cross and other bracing rings must be adequate and placed as quickly as excavation permits.

Sheet piling is generally chosen for its efficiency, versatility, and economy. Cofferdam sheet piling and any internal bracing are designed by the Contractor, with the design being accepted by the Department. Other forms of temporary sheet piling are designed by the Department. Temporary sheet piling is not the same as temporary shoring. Temporary shoring is designed by the Contractor and may involve sheet piling or other forms of excavation support.

14.10.2 Sheet Piling Materials

Although sheet piling can be composed of timber or precast concrete members, these material types are seldom, if ever, used on Wisconsin transportation projects.

Steel sheet piles are by far the most extensively used type of sheeting in temporary construction because of their availability, various sizes, versatility and ability to be reused. Also, they are very adaptable to permanent structures such as bulkheads, seawalls and wharves if properly protected from salt water.

Sheet pile shapes are generally Z, arched or straight webbed. The Z and the medium to high arched sections have high section moduli and can be used for substantial cantilever lengths or relatively high lateral pressures. The shallow arched and straight web sections have high interlocking strength and are employed for cellular cofferdams. The Z-section has a ball-and-socket interlock and the arched and straight webbed sections have a thumb-and-finger interlock capable of swinging 10 degrees. The thumb-and-finger interlock provides high tensile strength and considerable contact surface to prevent water passage. Continuous steel sheet piling is not completely waterproof, but does stop most water from passing through the joints. Steel sheet piling is usually 3/8 to 1/2 inch thick. Designers should specify the required section modulus and embedment depths on the plans, based on bending requirements and also account for corrosion resistance as appropriate.

Refer to steel catalogs for typical sheet pile sections. Contractors are allowed to choose either hot or cold rolled steel sections meeting the specifications. Previously used steel sheet piling may be adequate for some temporary situations, but should not be allowed on permanent applications.

14.10.3 Driving of Sheet Piling

All sheets in a section are generally driven partially to depth before all are driven to the final required depths. There is a tendency for sheet piles to lean in the direction of driving producing a net "gain" over their nominal width. Most of this "gain" can be eliminated if the piles are driven a short distance at a time, say from 6 feet to one third of their length before any single pile is driven to its full length. During driving if some sheet piles strike an obstruction, move to the next pile that can be driven and then return to the piles that resisted driving. With interlock guides on both sides and a heavier hammer, it may be possible to drive the obstructed sheet to the desired depth.

Sheet piles are installed by driving with gravity, steam, air or diesel powered hammers, or by vibration, jacking or jetting depending on the subsurface conditions, and pile type. A vibratory or double acting hammer of moderate size is best for driving sheet piles. For final driving of long heavy piles a single acting hammer may be more effective. A rapid succession of blows is generally more effective when driving in sand and gravel; slower, heavier blows are better for penetrating clay materials. For efficiency and impact distribution, where possible, two sheets are driven together. If sheets adjacent to those being driven tend to move down below the required depth, they are stopped by welding or bolting to the guide wales. When sheet piles are pulled down deeper than necessary by the driving of adjacent piles, it is generally better to fill in with a short length at the top, rather than trying to pull the sheet back up to plan location.

14.10.4 Pulling of Sheet Piling

Vibratory hammers are most effective in removing sheets and typically used. Sheet piles are pulled with air or steam powered extractors or inverted double acting hammers rigged for this application. If piles are difficult to pull, slight driving is effective in breaking them loose. Pulled sheet piling is to be handled carefully since they may be used again; perhaps several times.

14.10.5 Design Procedure for Sheet Piling Walls

A description of sheet pile design is given in **LRFD [11.8.2]** as **"**Cantilevered Wall Design" along with the earth pressure diagrams showing some simplified earth pressures. They are also referred to as flexible cantilevered walls. Steel sheet pile walls can be designed as cantilevered walls up to approximately 15 feet in height. Over 15 feet height, steel sheet pile walls may require tie-backs with either prestressed soil anchors, screw anchors, or deadman-type anchors.

The preferred method of designing cantilever sheet piling is by the "Conventional Method" as described in the *United States Steel Sheet Piling Design Manual* (February,1974). The

Geotechnical Engineer provides the soil design parameters including cohesion values, angles of internal friction, wall friction angles, soil densities, and water table elevations. The lateral earth pressures for non gravity cantilevered walls are presented in **LRFD [3.11.5.6].**

Anchored wall design must be in accordance with **LRFD [11.5.6]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

All areas of permanent exposed steel sheet piling above the ground line shall be coated or painted prior to driving. Corrosion potential should be considered in all steel sheet piling designs. Special consideration should be given to permanent steel sheet piling used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see *Facilities Development Manual*, Procedure 13-1-15).

Permanent sheet pile walls below the watertable may require the use of composite strip drains, collector and drainage pipes before placement of the final concrete facing.

The appearance of permanent steel sheet piling walls may be enhanced by applying either precast concrete panels or cast-in-place concrete surfacing. Welded stud-shear connectors can be used to attach cast-in-place concrete to the sheet piling. Special surface finishes obtained by using form liners or other means and concrete stain or a combination of stain and paint can be used to enhance the concrete facing aesthetics.

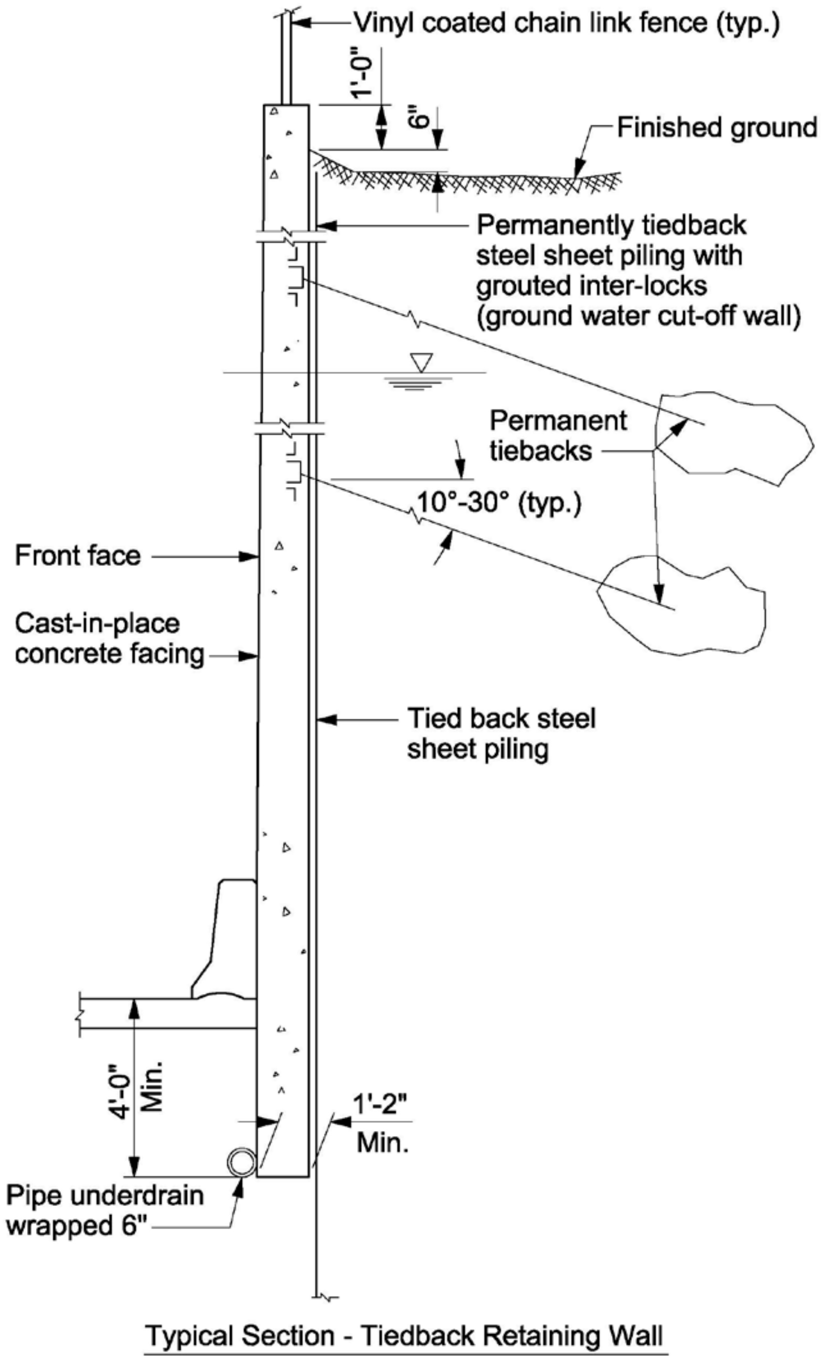


Figure 14.10-1
Typical Anchored Sheet Pile Wall

14.10.6 Summary of Design Requirements

1. Load and Resistance Factor

Load Combination	Load Factors	Resistance Factor
Strength I (maximum)	EH-Horizontal Earth Pressure: δ =1.50 **LRFD [Table 3.4.1-2]**	-----------
Strength I (maximum)	LS-Live Load Surcharge: δ =1.75 **LRFD [Table 3.4.1-1]**	-----------
Strength I (maximum)	-----------	Passive resistance of vertical elements: ϕ=0.75 **LRFD [Table11.5.7-1]**
Service I	-----------	Overall Stability: ϕ=0.75, when geotechnical parameters are well defined, and the slope does not support or contain a structural element
Service I	-----------	Overall Stability: ϕ=0.65, when geotechnical parameters are based on limited information, or the slope does support or contain a structural element

Table 14.10-1
Summary of Design Requirements

2. Foundation design parameters

   Use values provided by the Geotechnical Engineer of record for permanent sheet pile walls. Temporary sheet pile walls are the Contractor's responsibility.

3. Traffic surcharge

   - Traffic live load surcharge = 240 lb/ft^2 or determined by site condition.
   - If no traffic live load is present, use 100 lb/ft^2 live load for construction equipment

4. Retained soil

   - Unit weight = 120 lb/ft^3
   - Angle of internal friction as determined from the Geotechnical Report.

5. Soil pressure theory

   Coulomb Theory.

6. Design life for anchorage hardware

   75 years minimum

7. Steel design properties

   Minimum yield strength = 39,000 psi

14.11 Soldier Pile Walls

Soldier pile walls are comprised of discrete vertical elements (usually steel H piles) and facing members (temporary and/or permanent) that extend between the vertical elements.

14.11.1 Design Procedure for Soldier Pile Walls

LRFD [11.8] Non-Gravity Cantilevered Walls covers the design of soldier pile walls. A simplified earth pressure distribution diagram is shown in **LRFD [3.11.5.6]** for permanent soldier pile walls. Another method that may be used is the "Conventional Method" or “Simplified Method” as described in "*United States Steel Sheet Piling Design Manual*", February, 1974. This method must be modified for the fact that it is based on continuous vertical wall elements whereas, soldier pile walls have discrete vertical wall elements. Using "Broms" method for designing drilled shafts is also acceptable.

The maximum spacing between vertical supporting elements (piles) depends on the wall height and the design parameters of the foundation soil. Spacing of 6 to 12 feet is typical. The piles are set in drilled holes and concrete is placed in the hole after the post is set. The pile system must be designed to handle maximum bending moment along length of embedded shaft. The maximum bending moment at any level in the facing can be determined from formulas in **LRFD [11.8.5.1]**. The minimum structural thickness on wall facings shall be 6 inches for precast panels and 10 inches with cast-in-place concrete.

The diameter of the drilled shaft is also dependent on the wall height and the design parameters of the foundation soil. The larger the diameter of the drilled shaft the smaller will be the required embedment of the shaft. The designer should try various shaft diameters to optimize the cost of the drilled shaft considering both material cost and drilling costs. Note that drilling costs are a function of both hole diameter and depth.

If the vertical elements are steel they shall be shop painted. Wall facings are usually given a special surface treatment created by brooming or tining vertically, using form liners, or using a pattern of rustication strips. The portion of the panel receiving the special treatment may be recessed, forming a border around the treated area. Concrete paints or stains may be used for color enhancements. When panel heights exceed 15 feet anchored walls may be needed. Anchored wall design must be in accordance with **LRFD [11.9]**. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

The concrete for soldier pile walls shall have a 28 day compressive strength of 4000 psi if non-prestressed and 5000 psi if prestressed except for the drilled shafts. Concrete for the drilled shafts shall have a 28 day compressive strength of 3500 psi. Reinforcement shall be uncoated Grade 60 in drilled shafts. In lieu of drainage aggregate a membrane may be used to seal the joints between the vertical elements and concrete panels to prevent water leakage. The front face of soldier pile walls shall be battered 1/4" per foot to account for short and long term deflection.

14.11.2 Summary of Design Requirements

Requirements

1. Resistance Factors

    - Overall Stability= 0.65 to 0.75 (based on how well defined the geotechnical parameters are and the support of structural elements)
    - Passive Resistance of vertical Elements = 0.75

2. Foundation Design Parameters

    Use values provided by the Geotechnical Engineer (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

3. Concrete Design Data

    - f'_c = 3500 psi (for drilled shafts)
    - f'_c = 4000 psi (non-prestressed panel)
    - f'_c = 5000 psi (prestressed panel)
    - f_y = 60,000 psi

4. Load Factors

    - Vertical earth pressure = 1.5
    - Lateral earth pressure = 1.5
    - Live load surcharge = 1.75

5. Traffic Surcharge

    - Traffic live load surcharge = 2 feet = 240 lb/ft^2
    - If no traffic surcharge, use 100 lb/ft^2

6. Retained Soil

    Use values provided by the Geotechnical Engineer (unit weight, angle of internal friction, and cohesion). Both drained and undrained parameters shall be considered.

7. Soil Pressure Theory

    Rankine's Theory or Coulombs Theory at the discretion of the designer.

8. Design Life for Anchorage Hardware

   75 year minimum

9. Steel Design Properties (H-piles)

   Minimum yield strength = 50,000 psi

14.12 Temporary Shoring

This information is provided for guidance. Refer to the *Facilities Development Manual* for further details.

Temporary shoring is used to support a temporary excavation or protect existing transportation facilities, utilities, buildings, or other critical features when safe slopes cannot be made for structural excavations. Shoring may be required within the limits of structures or on the approach roadway due to grade changes or staged construction. Temporary shoring generally includes non-anchored temporary sheet piles, temporary soldier pile walls, temporary soil nails, cofferdam, or temporary mechanically stabilized earth (MSE) walls.

Temporary shoring is designed by the contractor. Shoring should not be required nor paid for when used primarily for the convenience of the contractor.

14.12.1 When Slopes Won't Work

Typically shoring will be required when safe slopes cannot be made due to geometric constraints of existing and proposed features within the available right-of-way. Occupation and Healthy Safety Administration (OSHA) requirements for temporary excavation slopes vary from a 1H:1V to a 2H:1V. The contractor is responsible for determining and constructing a safe slope based on actual site conditions.

In most cases, the designer can assume that an OSHA safe temporary slope can be cut on a 1.5H:1V slope; however other factors such as soil types, soil moisture, surface drainage, and duration of excavation should also be factored into the actual slope constructed. As an added safety factor, a 3-foot berm should be provided next to critical points or features prior to beginning a 1.5H:1V slope to the plan elevation of the proposed structure. Sufficient room should be provided adjacent to the structure for forming purposes (typically 2-3 feet).

14.12.2 Plan Requirements

Contract plans should schematically show in the plan and profile details all locations where the designer has determined that temporary shoring will be required. The plans should note the estimated length of the shoring as well as the minimum and maximum required height of exposed shoring. These dimensions will be used to calculate the horizontal projected surface area projected on a vertical plane of the exposed shoring face.

14.12.3 Shoring Design/Construction

The Contractor is responsible for design, construction, maintenance, and removal of the temporary shoring system in a safe and controlled manner. The adequacy of the design should be determined by a Wisconsin Professional Engineer knowledgeable of specific site conditions and requirements. The temporary shoring should be designed in accordance with the requirements described in 14.4.2 and 14.4.3. A signed and sealed copy of proposed designs must be submitted to the WisDOT Project Engineer for information.

14.13 Noise Barrier Walls

14.13.1 Wall Contract Process

WisDOT has classified all noise walls (both proprietary and non-proprietary) into three wall systems. All proprietary systems must be pre-approved prior to being considered for use on WisDOT projects. The three noise wall systems that are considered for WisDOT projects include the following:

1. Double-sided sound absorptive noise barriers

2. Single-sided sound absorptive noise barriers

3. Reflective noise barriers

If a wall is required, the designer must determine which wall system or systems are suitable for a given wall location. In some locations all wall systems may be suitable, whereas in other locations some wall systems may not be suitable. Information on aesthetic qualities and special finishes and colors of proprietary systems is available from the manufacturers. Information on approved concrete paints, stains and coatings is also available from the Structures Design Section. Designers are encouraged to contact the Structures Design Section (608-266-8494) if they have any questions about the material presented in the *Bridge Manual*.

The step by step process required to select a suitable wall system or systems for a given wall location is as follows:

Step 1: Investigate alternatives

Investigate alternatives to walls such as berms, plantings, etc.

Step 2: Geotechnical analysis

If a wall is required, geotechnical personnel shall conduct a soil investigation at the wall location and determine soil design parameters for the foundation soil. Geotechnical personnel are also responsible for recommending remedial methods of improving soil bearing capacity if required.

Step 3: Evaluate basic wall restrictions

The designer shall examine the list of suitable wall systems using the Geotechnical Report and remove any system that does not meet usage restrictions for the site.

Step 4: Determine suitable wall systems

The designer shall further examine the list of suitable wall systems for conformance to other considerations. Refer to Chapter 2 – General and Chapter 6 – Plan Preparation for a discussion on aesthetic considerations.

14.14 Contract Plan Requirements

The following minimum information shall be required on the plans.

1. Finish grades at rear and front of wall at 25 foot intervals or less.

2. Final cross sections as required for wall designer.

3. Beginning and end stations of wall and offsets from reference line to front face of walls. If reference line is a horizontal curve give offsets from a tangent to the curve.

4. Location of right-of-way boundaries, and construction easements relative to the front face of the walls.

5. Location of utilities if any and indicate whether to remain in place or be relocated or abandoned.

6. Special requirements on top of wall such as copings, railings, or traffic barriers.

7. Footing or leveling pad elevations if different than standard.

8. General notes on standard insert sheets.

9. Soil design parameters for retained soil, backfill soil and foundation soil including angle of internal friction, cohesion, coefficient of sliding friction, groundwater information and ultimate and/or allowable bearing capacity for foundation soil. If piles are required, give skin friction values and end bearing values for displacement piles and/or the allowable steel stress and anticipated driving elevation for end bearing piles.

10. Soil borings.

11. Details of special architectural treatment required for each wall system.

12. Wall systems, system or sub-systems allowed on projects.

13. Abutment details if wall is component of an abutment.

14. Connection and/or joint details where wall joins another structure.

15. Groundwater elevations.

16. Drainage provisions at heel of wall foundations.

17. Drainage at top of wall to divert run-off water.

18. Location of name plate.

14.15 Construction Documents

14.15.1 Bid Items and Method of Measurement

Proprietary retaining walls shall include all required bid items necessary to build the wall system provided by the contractor. The unit of measurement shall be square feet and shall include the exposed wall area between the footing and the top of the wall measured to the top of any copings. For setback walls the area shall be based on the walls projection on a vertical plane. The bid item includes designing the walls preparing plans, furnishing and placing all materials, including all excavations, temporary bracing, piling, (including delivered and driven), poured in place or precast concrete or blocks, leveling pads, soil reinforcement systems, structural steel, reinforcing steel, backfills and infills, drainage systems and aggregate, geotextiles, architectural treatment including painting and/or staining, drilled shafts, wall toppings unless excluded by contract, wall plantings, joint fillers, and all labor, tools, equipment and incidentals necessary to complete the work.

The contractor will be paid for the plan quantity as shown on the plans. (The intent is a lump sum bid item but is bid as square feet of wall). The top of wall coping is any type of cap placed on the wall. It does not include any barriers. Measurement is to the bottom of the barrier when computing exposed wall area.

Non-proprietary retaining walls are bid based on the quantity of materials used to construct the wall such as concrete, reinforcing steel, piling, etc. These walls are:

- Cast-in-Place Concrete Cantilever Walls
- Soldier Pile Walls
- Steel Sheet Piling Walls

14.15.2 Special Provisions

The Structures Design Section has Standard Special Provisions for:

- Wall Modular Block Gravity LRFD, Item SPV.0165.
- Wall Modular Block Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165
- Wall Concrete Panel Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165
- Wall CIP Facing Mechanically Stabilized Earth LRFD, Item SPV.0165.
- Wall Wire Faced Mechanically Stabilized Earth LRFD/QMP, Item SPV.0165.
- Temporary Wall Wire Faced Mechcanically Stabilized Earth LRFD/QMP, Item SPV.0165
- *Wall Gabion LRFD, SPV under development.*

- *Wall Modular Bin or Crib LRFD, SPV under development.*

Note that the use of QMP Special Provisions began with the December 2014 letting or prior to December 2014 letting at the Region's request.

The designer determines what wall systems(s) are applicable for the project. The approved names of suppliers are inserted for each eligible wall system. The list of approved proprietary wall suppliers is maintained by the Bureau of Structures which is responsible for the Approval Process for earth retaining walls, 14.16.

14.16 Submittal Requirements for Pre-Approval Process

14.16.1 General

The following four wall systems require the supplier or manufacturer to submit to the Structural Design Section a package that addresses the items specified in paragraph C.

1. Modular Block Gravity Walls
2. MSE Walls with Modular Block Facings
3. MSE Walls with Precast Concrete Panel Facings
4. Modular Concrete Bin or Crib Walls

14.16.2 General Requirements

Approval of retaining wall systems allows for use of these systems on Wisconsin Department of Transportation (WisDOT) projects upon the manufacturer's certification that the system as furnished to the contractor (or purchasing agency) complies with the design procedures specified in the *Bridge Manual*. WisDOT projects include: State, County and Municipal Federal Aid and authorized County and Municipal State Aid projects in addition to materials purchased directly by the state.

The manufacturer shall perform all specification tests with qualified personnel and maintain an acceptable quality control program. The manufacturer shall maintain records of all its control testing performed in the production of retaining wall systems. These test records shall be available at all times for examination by the Construction Materials Engineer for Highways or designee. Approval of materials will be contingent upon satisfactory compliance with procedures and material conformance to requirements as verified by source and field samples. Sampling will be performed by personnel during the manufacture of project specific materials.

14.16.3 Qualifying Data Required For Approval

Applicants requesting Approval for a specific system shall provide three copies of the documentation showing that they comply with *AASHTO LRFD* and *WisDOT Standard Specifications* and the design criteria specified in the *Bridge Manual*.

1. An overview of the system, including system theory.
2. Laboratory and field data supporting the theory.
3. Detailed design procedures, including sample calculations for installations with no surcharge, level surcharge and sloping surcharge.
4. Details of wall elements, analysis of structural elements, capacity demand ratio, load and resistance factors, estimated life, corrosion design procedure for soil

reinforcement elements, procedures for field and laboratory evaluation including instrumentation and special requirements, if any.

5. Sample material and construction control specifications - showing material type, quality, certifications, field testing and placement procedures.

6. A well documented field construction manual describing in detail and with illustrations where necessary, the step by step construction sequence.

7. Details for mounting a concrete traffic barrier on the wall adjoining both concrete and flexible pavements (if applicable).

8. Pullout data for facing block/geogrid connection and soil pullout data (if applicable).

9. Submission of practical application with photos for all materials, surface textures and colors representative of products being certified.

10. Submission, if requested, to an on-site production process control review, and record keeping review.

11. List of installations including owner name and wall location.

12. Limitations of the wall system.

The above materials may be submitted at any time (recommend a minimum of 15 weeks) but, to be considered for a particular WisDOT project, must be approved prior to the bid opening date. The material should be clearly detailed and presented according to the prescribed outline.

After final review and approval of comments with the Bureau of Structures, the manufacturer will be approved to begin presenting the system on qualified projects.

14.16.4 Maintenance of Approval Status as a Manufacturer

The supplier or manufacturer must request to be reapproved bi-annually. The request shall be in writing and certify that the plant production process control and materials testing and design procedures haven't changed since the last review. The request shall be received within two years of the previous approval or the approval status will be terminated. Upon request for re-approval an on-site review of plant process control and materials testing may be conducted by WisDOT personnel. Travel expenses for trips outside the State of Wisconsin involved with this review will be borne by the manufacturer.

For periodic on-site reviews, access to the plant operations and materials records shall be provided to a representative of the Construction Materials Engineer during normal working hours upon request.

If the supplier or manufacturer introduces a new material, or cross-section, or a new design procedure, into its product line, the new feature must be submitted for approval. If the new feature/features are significantly different from the original product, the new product may be subjected to a complete review for approval.

14.16.5 Loss of Approved Status

Approval to deliver the approved system may be withdrawn under the following conditions:

Design Conformance

1. Construction does not follow design procedures.
2. Incorrect design procedures are used on projects.

Materials

3. Inability to consistently supply material meeting specification.
4. Inability to meet test method precision limits for quality control testing.
5. Lack of maintenance of required records.
6. Improper documentation of shipments.
7. Not maintaining an acceptable quality control program.

The decision to remove approval from a manufacturer on a specific system rests with the Construction Materials Engineer for Highways or the State Bridge Engineer.

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E14-1 Cast-In-Place Concrete Cantilever Wall on Spread Footing, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on a spread footing conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. ***(Example is current through LRFD Seventh Edition - 2015 Interim)***

Sample design calculations for bearing resistance, external stability (sliding, eccentricity and bearing) and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-1.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-1.1-1 will be designed appropriately to accommodate a State Trunk Highway. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

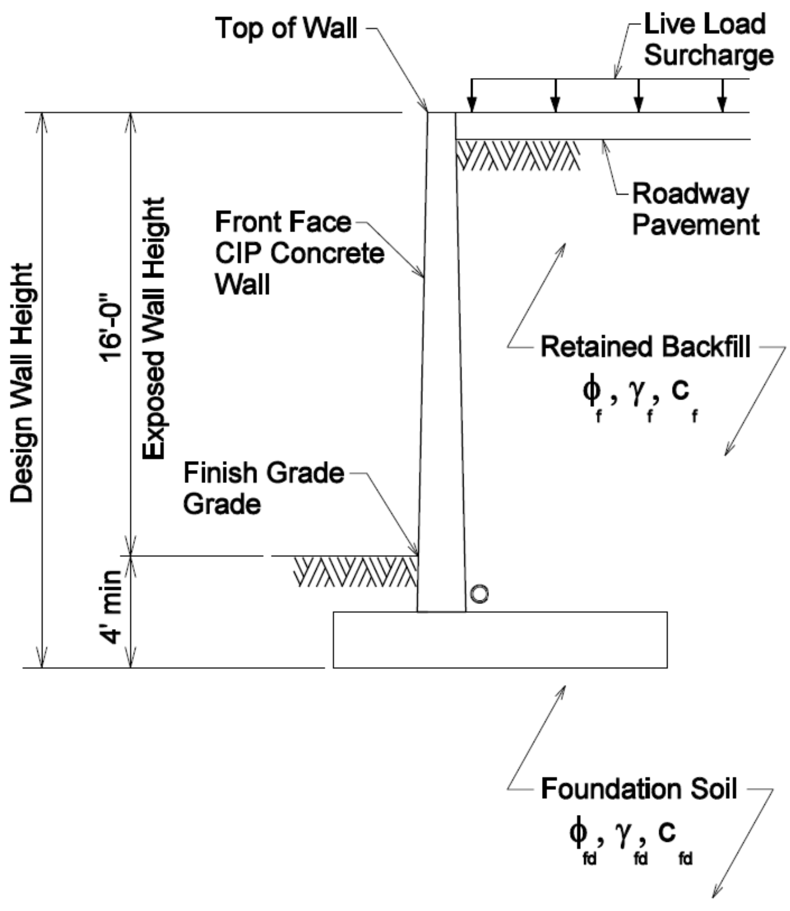

Figure E14-1.1-1
CIP Concrete Wall Adjacent to Highway

E14-1.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 30\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit weight, kcf

$c_f = 0$ Cohesion, ksf

$\delta = 21\ deg$ Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

Foundation Soil Design Parameters

$\phi_{fd} = 34\ deg$ Angle of internal friction

$\gamma_{fd} = 0.120$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, ksf

Reinforced Concrete Parameters

$f'_c = 3.5$ Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $w_c = \gamma_c$ Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ Modulus of elasticity of concrete, ksi **LRFD [C5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 1.0$ Distance from wall backface to edge of traffic, ft

$\frac{H}{2} = 10.00$ Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet)

Shall live load surcharge be included? check = "YES"

$h_{eq} = 2.0$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

Pavement Parameters

$\gamma_p = 0.150$ Pavement unit weight, kcf

Resistance Factors

$\phi_b = 0.55$ Bearing resistance (gravity and semi-gravity walls) **LRFD [Table 11.5.7-1]**

$\phi_s = 1.00$ Sliding resistance **LRFD [Table 11.5.7-1]**

$\phi_\tau = 1.00$ Sliding resistance (shear resistance between soil and foundation) **LRFD [Table 11.5.7-1]**

$\phi_{ep} = 0.50$ Sliding resistance (passive resistance) **LRFD [Table 10.5.5.2.2-1]**

$\phi_F = 0.90$ Concrete flexural resistance (Assuming tension-controlled) **LRFD [5.5.4.2.1]**

$\phi_v = 0.90$ Concrete shear resistance **LRFD [5.5.4.2.1]**

E14-1.6 Evaluate External Stability of Wall

Three potential external failure mechanisms will be considered in this example. These failures include bearing, limiting eccentricity and sliding. Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-1.6.1 Bearing Resistance at Base of the Wall

The following calculations are based on **Strength Ib**:

Compute resultant location (distance from Point 'O' Figure E14-1.4-3)

$\Sigma M_R = MV\_Ib$ $\qquad \Sigma M_R = 205.8$ kip-ft/ft

$\Sigma M_O = MH\_Ib$ $\qquad \Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = V\_Ib$ $\qquad \Sigma V = 29.3$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ Distance from Point "O" the resultant intersects the base

$\boxed{x = 4.25}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x$$ $\boxed{e = 0.75}$ ft

Note: The vertical stress is assumed to be uniformly distributed over the effective bearing width, B', since the wall is supported by a soil foundation **LRFD [11.6.3.2]**. The effective bearing width is equal to B-2e. When the foundation eccentricity is negative the actual bearing width, B, will be used.

Compute the ultimate bearing stress

$$\sigma_V = \frac{\Sigma V}{B - 2\,e}$$ $\boxed{\sigma_V = 3.44}$ ksf/ft

Factored bearing resistance

$\boxed{q_R = 5.64}$ ksf/ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Bearing1} = \frac{q_R}{\sigma_V}$$ $\boxed{CDR_{Bearing1} = 1.64}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

E14-1.6.2 Limiting Eccentricity at Base of the Wall

The location of the resultant of the reaction forces is limited to the middle two-thirds o base width for a soil foundation (i.e., e_{max} = B/3). The following calculations are base **Strength Ia**:

Maximum eccentricity

$$e_{max} = \frac{B}{3}$$ $\boxed{e_{max} = 3.33}$ ft

Compute resultant location (distance from Point 'O' Figure E14-1.4.3)

$\Sigma M_R = MV\_Ia$ $\Sigma M_R = 150.0$ kip-ft/ft

$\Sigma M_O = MH\_Ia$ $\Sigma M_O = 81.3$ kip-ft/ft

$\Sigma V = V\_Ia$ $\Sigma V = 20.9$ kip/ft

$$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$ Distance from Point "O" the resultant intersects the base

$\boxed{x = 3.29}$ ft

Compute the wall eccentricity

$$e = \frac{B}{2} - x$$ $\boxed{e = 1.71}$ ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Eccentricity1} = \frac{e_{max}}{e}$$ $\boxed{CDR_{Eccentricity1} = 1.94}$

Is the $CDR \geq 1.0$? $\boxed{check = "OK"}$

E14-1.6.3 Sliding Resistance at Base of the Wall

For sliding failure, the horizontal force effects, R_u, is checked against the sliding resistance, R_R, where $R_R=\phi R_n$ **LRFD [10.6.3.4]**. If sliding resistance is not adequate a shear key will be investigated. The following calculations are based on **Strength Ia**:

Factored Sliding Force, R_u

$R_u = H\_Ia$ $\boxed{R_u = 11.7}$ kip/ft

Sliding Resistance, R_R

$$R_R = \phi_s R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

Compute sliding resistance between soil and foundation, $\phi_T\ R_T$

$\Sigma V = V\_Ia$ $\boxed{\Sigma V = 20.9}$ kip/ft

$R_T = \Sigma V \tan(\phi_{fd})$ $\boxed{R_T = 14.1}$ kip/ft

$\phi_T = 1.00$ $\boxed{\phi_T\ R_T = 14.1}$ kip/ft

Compute passive resistance throughout the design life of the wall, $\phi_{ep}\ R_{ep}$

$r_{ep1} = k_p\ \gamma_{fd}\ y_1$ Nominal passive pressure at y_1 $\boxed{r_{ep1} = 1.70}$ kip/ft

$r_{ep2} = k_p\ \gamma_{fd}\ y_2$ Nominal passive pressure at y_2 $\boxed{r_{ep2} = 2.12}$ kip/ft

$R_{ep} = \dfrac{r_{ep1} + r_{ep2}}{2}(y_2 - y_1)$ $\boxed{R_{ep} = 1.9}$ kip/ft

$\phi_{ep} = 0.50$ $\boxed{\phi_{ep}\ R_{ep} = 1.0}$ kip/ft

Compute nominal resistance against failure by sliding, R_n

$R_n = \phi_T\ R_T + \phi_{ep}\ R_{ep}$ $\boxed{R_n = 15.1}$ kip/ft

Compute factored resistance against failure by sliding, R_R

$\phi_s = 1.00$

$R_R = \phi_s\ R_n$ $\boxed{R_R = 15.1}$ kip/ft

Capacity:Demand Ratio (CDR)

$CDR_{Sliding1} = \dfrac{R_R}{R_u}$ $\boxed{CDR_{Sliding1} = 1.29}$

Is the CDR ≥ 1.0 ? $\boxed{check = \text{"OK"}}$

E14-1.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. The critical sections for flexure are taken at the front, back and bottom of them stem. For simplicity, critical sections for shear will be taken at the critical sections used for flexsure. In actuality, the toe and stem may be designed for shear at the effective depth away from the face. Crack control and temperature and shrinkage considerations will also be included.

E14-1.7.1 Evaluate Heel Strength

Analyze heel requirements.

E14-1.7.1.1 Evaluate Heel Shear Strength

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B}V_4 + V_6\right) + 1.35\left(V_7 + V_8 + V_9\right) + 1.75\left(V_{10}\right) + 1.50\left(V_{11}\right)$$

$\boxed{V_u = 21.9}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\,\beta\sqrt{f'_c}\,b_v\,d_v$

$V_{n2} = 0.25\,f'_c\,b_v\,d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 7.0$ in (bar spacing)

$Bar_{No} = 6$ (transverse bar size)

$Bar_D = 0.750$ in (transverse bar diameter)

$Bar_A = 0.440$ in^2 (transverse bar area)

$\alpha_1 = 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$

$\boxed{A_s = 0.75}$ in^2/ft

$$d_s = D\,12 - cover - \frac{Bar_D}{2}$$

$\boxed{d_s = 21.6}$ in

$$a = \frac{A_s\,f_y}{\alpha_1\,f'_c\,b}$$

$\boxed{a = 1.3}$ in

$\boxed{1.33\ M_u = 63.7}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$? $\boxed{check = \text{"OK"}}$

E14-1.7.2 Evaluate Toe Strength

The structural design of the footing toe is calculated using a linear contact stress distribution for bearing for all soil and rock conditions.

E14-1.7.2.1 Evaluate Toe Shear Strength

For **Strength Ib**:

$\Sigma M_R = MV\_Ib$ $\boxed{\Sigma M_R = 205.8}$ kip-ft/ft

$\Sigma M_O = MH\_Ib$ $\boxed{\Sigma M_O = 81.3}$ kip-ft/ft

$\Sigma V = V\_Ib$ $\boxed{\Sigma V = 29.3}$ kip/ft

$x = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$ $\boxed{x = 4.3}$ ft

$e = \max\left(0, \frac{B}{2} - x\right)$ $\boxed{e = 0.75}$ ft

$\sigma_{max} = \frac{\Sigma V}{B}\left(1 + 6\frac{e}{B}\right)$ $\boxed{\sigma_{max} = 4.24}$ ksf/ft

$\sigma_{min} = \frac{\Sigma V}{B}\left(1 - 6\frac{e}{B}\right)$ $\boxed{\sigma_{min} = 1.62}$ ksf/ft

Calculate the average stress on the toe

$\sigma_v = \frac{\sigma_{max} + \left[\sigma_{min} + \frac{B - A}{B}\left(\sigma_{max} - \sigma_{min}\right)\right]}{2}$ $\boxed{\sigma_v = 3.78}$ ksf/ft

$V_u = \sigma_v A$ $\boxed{V_u = 13.2}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

in which: $V_c = 0.0316\ \beta\sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Design footing toe for shear

$cover = 3.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 5$ (transverse bar size)

$Bar_D = 0.63$ in (transverse bar diameter)

$Bar_A = 0.31$ in² (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 0.41} \text{ in}^2\text{/ft}$$

$$d_s = D\ 12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 20.7} \text{ in}$$

$$a = \frac{A_s\ f_y}{\alpha_1\ f'_c\ b} \qquad \boxed{a = 0.7} \text{ in}$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 20.3} \text{ in}$$

$$d_{v2} = 0.9\ d_s \qquad \boxed{d_{v2} = 18.6} \text{ in}$$

$$d_{v3} = 0.72\ D\ 12 \qquad \boxed{d_{v3} = 17.3} \text{ in}$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 20.3} \text{ in}$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$

$$V_c = 0.0316\ \beta \sqrt{f'_c}\ b\ d_v$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 28.9} \text{ kip/ft}$$

$$V_{n2} = 0.25\ f'_c\ b\ d_v \qquad \boxed{V_{n2} = 213.6} \text{ kip/ft}$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 28.9} \text{ kip/ft}$$

$$V_r = \phi_v\ V_n \qquad \boxed{V_r = 26.0} \text{ kip/ft}$$

$$\boxed{V_u = 13.2} \text{ kip/ft}$$

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 10.0$ in (bar spacing)

$Bar_{No} = 8$ (transverse bar size)

$Bar_D = 1.00$ in (transverse bar diameter)

$Bar_A = 0.79$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ $\boxed{A_s = 0.95}$ in^2/ft

$$d_s = T_b\,12 - cover - \frac{Bar_D}{2}$$ $\boxed{d_s = 23.0}$ in

$$a = \frac{A_s\,f_y}{\alpha_1\,f'_c\,b}$$ $\boxed{a = 1.6}$ in

$$d_{v1} = d_s - \frac{a}{2}$$ $\boxed{d_{v1} = 22.2}$ in

$$d_{v2} = 0.9\,d_s$$ $\boxed{d_{v2} = 20.7}$ in

$$d_{v3} = 0.72\,T_b\,12$$ $\boxed{d_{v3} = 18.4}$ in

$$d_v = \max(d_{v1}, d_{v2}, d_{v3})$$ $\boxed{d_v = 22.2}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$

$$V_c = 0.0316\,\beta\,\sqrt{f'_c}\,b\,d_v$$

$$V_{n1} = V_c$$ $\boxed{V_{n1} = 31.5}$ kip/ft

$$V_{n2} = 0.25\,f'_c\,b\,d_v$$ $\boxed{V_{n2} = 233.1}$ kip/ft

$$V_n = \min(V_{n1}, V_{n2})$$ $\boxed{V_n = 31.5}$ kip/ft

$$V_r = \phi_v\,V_n$$ $\boxed{V_r = 28.4}$ kip/ft

$\boxed{V_u = 9.6}$ kip/ft

Is V_u less than V_r? $\boxed{check = "OK"}$

E14-1.7.3.2 Evaluate Stem Flexural Strength at Footing

$M_u = M_{u1}$ $\boxed{M_u = 60.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$M_n = A_s\, f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$ $\boxed{M_n = 105.2}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \frac{a}{\beta_1}$ $\boxed{c = 1.87}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F\, M_n$ $\boxed{M_r = 94.7}$ kip-ft/ft

$\boxed{M_u = 60.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.24\sqrt{f'_c}$ $\boxed{f_r = 0.45}$ ksi

$I_g = \frac{1}{12}\, b\, (T_b\, 12)^3$ $\boxed{I_g = 16581}$ in^4

$y_t = \frac{1}{2}\, T_b\, 12$ $\boxed{y_t = 12.8}$ in

$S_c = \frac{I_g}{y_t}$ $\boxed{S_c = 1301}$ in^3

Check the maximum spacing requirements

$s_1 = \min(3\ h_s, 18)$ $\boxed{s_1 = 18.0}$ in

$s_2 = \begin{cases} 12 & \text{if } h_s > 18 \\ s_1 & \text{otherwise} \end{cases}$ For walls and footings (in) $\boxed{s_2 = 18.0}$ in

$s_{max} = \min(s_1, s_2)$ $\boxed{s_{max} = 18.0}$ in

Is the bar spacing less than s_{max}? $\boxed{\text{check} = \text{"OK"}}$

E14-1.8 Summary of Results

List all summaries.

E14-1.8.1 Summary of External Stability

Based on the defined project parameters the following external stability checks have been satisfied:

External Check	CDR
	Strength
Sliding	1.29
Eccentricity	1.94
Bearing	1.64

Table E14-1.8-1
Summary of External Stability Computations

E14-1.8.2 Summary of Wall Strength Design

The required wall reinforcing from the previous computations are presented in Figure E14-1.9-1.

E14-1.8.3 Drainage Design

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are met by providing granular, free draining backfill material with a pipe underdrain located at the bottom of the wall (Assumed wall is adjacent to sidewalk) as shown in Figure E14-1.9-1.

E14-1.9 Final CIP Concrete Wall Schematic

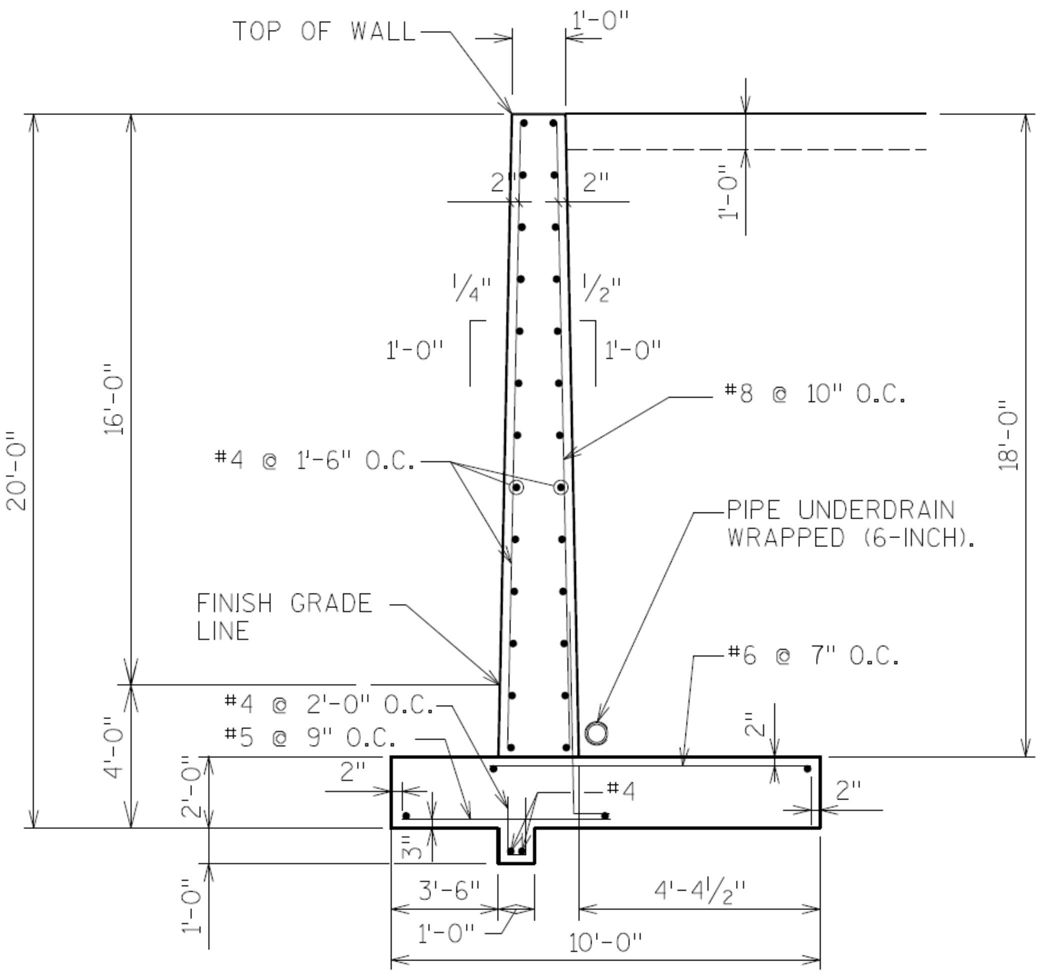

Figure E14-1.9-1
Cast-In-Place Wall Schematic

Summary of Factored Forces & Moments:

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	88.9	281.0	51.1	561.1
Strength Ib	111.6	295.0	51.1	561.1
Service I	80.9	200.7	33.8	369.6

Table E14-2.4-4
Summary of Factored Loads & Moments

E14-2.5 Evaluate External Stability of MSE Wall

Three potential external failure mechanisms will be considered in this example (sliding at the base, limiting eccentricity and bearing resistance). Global (overall) stability requirements are assumed to have been satisfied in prior calculations. Design calculations will be carried out for the governing limit states only.

E14-2.5.1 Sliding Resistance at Base of MSE Wall

The following calculations are based on **Strength Ia**:

Factored Sliding Force

$R_u = H\_Ia$ $\boxed{R_u = 51.14}$ kip/ft

Sliding Resistance

To compute the coefficient of sliding friction for discontinuous reinforcement use the lesser friction angle of the reinforced back fill, ϕ_r , or foundation soil, ϕ_{fd}, **LRFD [11.10.5.3]**.

$\phi_\mu = \min(\phi_r, \phi_{fd})$ $\boxed{\phi_\mu = 30 \text{ deg}}$

$\mu = \tan(\phi_\mu)$ $\boxed{\mu = 0.577}$

$V\_Ia = 88.9$ Factored vertical load, kip/ft

$V_{Nm} = \mu\ V\_Ia$ $\boxed{V_{Nm} = 51.3}$ kip/ft

$\phi_s = 1.0$

$R_R = \phi_s V_{Nm}$ $\boxed{R_R = 51.33}$ kip/ft

<u>Capacity:Demand Ratio (CDR)</u>

$CDR_{Sliding} = \dfrac{R_R}{R_u}$ $\boxed{CDR_{Sliding} = 1.00}$

Is the CDR ≥ 1.0 ? $\boxed{check = \text{"OK"}}$

E14-2.5.2 Limiting Eccentricity at Base of MSE Wall

The location of the resultant of the reaction forces is limited to the middle two-thirds of the base width for a soil foundation (i.e., e_{max} = L/3) **LRFD [11.6.3.3]**. The following calculations are based on **Strength Ia**:

Maximum eccentricity

$$e_{max} = \frac{L}{3}$$ $\boxed{e_{max} = 6.67}$ ft

Compute wall eccentricity (distance from Point 'O' in Figure E14-2.4-1)

$\Sigma M_R = MV\_Ia$ Summation of resisting moments for Strength Ia

$\Sigma M_O = MH\_Ia$ Summation of overturning moments for Strength Ia

$\Sigma V = V\_Ia$ Summation of vertical loads for Strength Ia

$\boxed{\Sigma M_R = 281.0}$ kip-ft/ft

$\boxed{\Sigma M_O = 561.1}$ kip-ft/ft

$\boxed{\Sigma V = 88.9}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$ $\boxed{e = 3.15}$ ft

<u>Capacity:Demand Ratio (CDR)</u>

$$CDR_{Eccentricity} = \frac{e_{max}}{e}$$ $\boxed{CDR_{Eccentricity} = 2.12}$

Is the $CDR \geq 1.0$? $\boxed{check = \text{"OK"}}$

Compute the design cross-sectional area of the reinforcement after sacrificial thicknesses have been accounted for during the wall design life per **LRFD [11.10.6.4.2a]**. The zinc coating life shall be calculated based on 0.58 mil/yr loss for the first 2 years and 0.16 mil/yr thereafter. After the depletion of the zinc coating, the steel design life is calculated and used to determine the sacrificial steel thickness after the steel design life. The sacrificial thickness of steel is based on 0.47 mil/yr/side loss.

Design_Life = Coating_Life + Steel_Design_Life = 75 years

$$Coating\_Life = 2 + \frac{Zinc - 2\ 0.58}{0.16} \qquad \boxed{Coating\_Life = 16.0} \text{ years}$$

$$Steel\_Design\_Life = Design\_Life - Coating\_Life \qquad \boxed{Steel\_Design\_Life = 59} \text{ years}$$

$$E_s = \left(\frac{0.47}{1000}\right) Steel\_Design\_Life\ (2) \qquad \boxed{E_s = 0.055} \text{ in}$$

$$E_c = E_n - E_s \qquad \boxed{E_c = 0.102} \text{ in}$$

$$Design\_Strip\_Area = E_c\ b \qquad \boxed{Design\_Strip\_Area = 0.201} \text{ in}^2$$

Compute the Factored Tensile Resistance, T_r

$$T_n = F_y\ Design\_Strip\_Area \qquad \boxed{T_n = 13.05} \text{ kip/strip}$$

$$T_r = \phi_t\ T_n \qquad \boxed{T_r = 9.79} \text{ kip/strip}$$

Determine the number of soil reinforcing strips based on tensile resistance, N_t

$$N_t = \frac{T_{max2}}{T_r} \qquad \boxed{N_t = 1.38} \text{ strips}$$

E14-2.6.5 Establish Number of Soil Reinforcing Strips at Z

$N_p = 1.48$ Based on pullout resistance, strips

$N_t = 1.38$ Based on tensile resistance, strips

Required number of strip reinforcements for each panel width (round up), N_g

$$N_g = ceil(max(N_t, N_p)) \qquad \boxed{N_g = 2} \text{ strips}$$

Calculate the horizontal spacing of reinforcement, S_h, at Z by dividing the panel width by the required number of strip reinforcements N_g.

$$S_h = \frac{w_p}{N_g} \qquad \boxed{S_h = 2.50} \text{ ft}$$

Note: The typical horizontal reinforcement spacing, S_h, will be provided at 2.5 ft. This will also be the maximum allowed spacing while satisfying the maximum spacing requirement of 2.7 ft. If the wall requires additional reinforcement the vertical spacing will be maintained and adjustments will be made to the horizontal spacing accordingly.

E14-2.7 Summary of Results

E14-2.7.1 Summary of External Stability

Based on the defined project parameters, embedment depth and length of reinforcement the following external stability checks have been satisfied:

External Check	CDR
Sliding	1.00
Eccentricity	2.12
Bearing	1.37

Table E14-2.7-1
Summary of External Stability Computations

E14-2.7.2 Summary of Internal Stability

Computations for the required number of strip reinforcements at each level is presented ir **Table E14-2.7-2**.

		Pullout			Rupture						
Layer	Z	σ_H	T_{max1}	P_{rr}	σ_H	T_{max2}	T_r	N_p	N_t	N_g	S_h
1	0.75	0.46	4.55	5.86	0.53	5.34	9.79	0.78	0.54	2	2.50
2	3.25	0.64	8.05	7.08	0.72	9.00	9.79	1.14	0.92	2	2.50
3	5.75	0.84	10.47	7.98	0.91	11.38	9.79	1.31	1.16	2	2.50
4	**8.25**	**1.01**	**12.67**	**8.54**	**1.08**	**13.55**	**9.79**	**1.48**	**1.38**	**2**	**2.50**
5	10.75	1.17	14.65	9.37	1.24	15.49	9.79	1.56	1.58	2	2.50
6	13.25	1.31	16.42	10.13	1.38	17.22	9.79	1.62	1.76	2	2.50
7	15.75	1.44	17.96	10.46	1.50	18.73	9.79	1.72	1.91	2	2.50
8	18.25	1.54	19.29	10.25	1.60	20.01	9.79	1.88	2.04	3	1.67
9	20.75	1.67	20.84	10.22	1.72	21.55	9.79	2.04	2.20	3	1.67

Table E14-2.7-2
Summary of Internal Stability Computation for Strength I Load Combinations

Summary of Factored Forces & Moments:

Load Combination	Vert. Loads V (kips/ft)	Moments MV (kip-kip/ft)	Horiz. Loads H (kips/ft)	Moments MH (kip-kip/ft)
Strength Ia	40.1	58.2	22.8	191.3
Strength Ib	52.0	63.6	22.8	191.3
Service I	38.1	43.9	14.8	123.3

Table E14-3.4-4
Summary of Factored Loads & Moments

E14-3.5 Evaluate External Stability of MSE Wall

Three potential external failure mechanisms will be considered in this example (sliding at the base, limiting eccentricity and bearing resistance). Overall (global) stability requirements are not included here. Design calculations will be carried out for the governing limit states only.

E14-3.5.1 Sliding Resistance at Base of MSE Wall

The following calculations are based on **Strength Ia**:

Factored Sliding Force

$R_u = H\_Ia$ $\boxed{R_u = 22.8}$ kip/ft

Sliding Resistance

To compute the coefficient of sliding friction for continuous reinforcement use the lesser friction angle of the reinforced back fill, ϕ_r , or the foundation soil, ϕ_{fd}, **LRFD [11.10.5.3]**.

$\phi_\mu = \min(\phi_r, \phi_{fd})$ $\boxed{\phi_\mu = 30 \text{ deg}}$

Note: Since continuous reinforcement is used, a slip plane may occur at the reinforcement layer. The sliding friction angle for this case shall use the lesser of (when applicable) ϕ_r, ϕ_{fd}, and ρ. Where ρ is the soil-reinforcement interface friction angle. Without specific data ρ may equal 2/3 ϕ_f with ϕ_f a maximum of 30 degrees. This check is not made in this example, but is required.

$\mu = \tan(\phi_\mu)$ $\boxed{\mu = 0.577}$

$V\_Ia = 40.1$ Factored vertical load, kip/ft

$V_{Nm} = \mu\ V\_Ia$ $\boxed{V_{Nm} = 23.1}$ kip/ft

$\phi_s = 1.00$

$R_R = \phi_s V_{Nm}$ $\boxed{R_R = 23.1}$ kip/ft

Capacity:Demand Ratio (CDR)

$$CDR_{Sliding} = \frac{R_R}{R_u}$$ $\boxed{CDR_{Sliding} = 1.02}$

Is the CDR ≥ 1.0 ? $\boxed{check = "OK"}$

E14-3.5.2 Limiting Eccentricity at Base of MSE Wall

The location of the resultant of the reaction forces is limited to the middle two-thirds of the base width for a soil foundation (i.e., e_{max} = L/3) **LRFD [11.6.3.3]**. The following calculations are based on **Strength Ia**.

Maximum eccentricity

$$e_{max} = \frac{L}{3}$$ $\boxed{e_{max} = 4.83}$ ft

Compute wall eccentricity (distance from Point 'O' Figure E14-3.4-1)

$\Sigma M_R = MV\_Ia$ Summation of resisting moments for Strength Ia

$\Sigma M_O = MH\_Ia$ Summation of overturning moments for Strength Ia

$\Sigma V = V\_Ia$ Summation of vertical loads for Strength Ia

$\boxed{\Sigma M_R = 58.2}$ kip-ft/ft

$\boxed{\Sigma M_O = 191.3}$ kip-ft/ft

$\boxed{\Sigma V = 40.1}$ kip/ft

$$e = \frac{\Sigma M_O - \Sigma M_R}{\Sigma V}$$ $\boxed{e = 3.32}$ ft

Capacity:Demand Ratio (CDR)

$$CDR_{Eccentricity} = \frac{e_{max}}{e}$$ $\boxed{CDR_{Eccentricity} = 1.46}$

Is the CDR ≥ 1.0 ? $\boxed{check = "OK"}$

E14-3.6.5 Establish Grade of Soil Reinforcing Elements at Each Level

Based on Pullout Resistance

$$CDR_{pullout} = \frac{P_{rr}}{T_{max1}}$$

$\boxed{CDR_{pullout} = 9.56}$

Is the CDR $\geq$ 1.0 ?

$\boxed{check = \text{"OK"}}$

Based on Tensile Resistance

$$CDR_{tensile} = \frac{T_r}{T_{max2}}$$

$\boxed{CDR_{tensile} = 1.24}$

Is the CDR $\geq$ 1.0 ?

$\boxed{check = \text{"OK"}}$

Note: If the wall requires additional reinforcement the vertical spacing will be maintained and adjustments will be made to the grade (strength) for each layer accordingly.

E14-3.7 Summary of Results

E14-3.7.1 Summary of External Stability

Based on the defined project parameters, embedment depth and length of reinforcement the following external stability checks have been satisfied:

External Check	CDR
Sliding	1.02
Eccentricity	1.46
Bearing	1.20

Table E14-3.7-1
Summary of External Stability Computations

E14-3.7.2 Summary of Internal Stability

Computations for the grades of geogrid reinforcements at each level is presented in Table E14-3.7-2.

Level	Z	Pullout σ_H	Pullout T_{max1}	Pullout P_{rr}	Rupture Grade	Rupture σ_H	Rupture T_{max2}	Rupture T_r	CDR_p	CDR_t
1	0.67	187	250	2455	#1	295	394	725	9.84	1.84
2	2.00	259	346	3280	#1	367	490	725	9.49	1.48
3	3.33	331	442	4221	#1	439	586	725	9.56	1.24
4	4.67	403	538	5280	#1	511	682	725	9.82	1.06
5	6.00	475	634	6456	#2	583	778	1449	10.19	1.86
6	7.33	547	730	7750	#2	655	874	1449	10.62	1.66
7	8.67	619	826	9161	#2	727	970	1449	11.10	1.49
8	10.00	691	922	10690	#2	799	1066	1449	11.60	1.36
9	11.33	763	1018	12336	#2	871	1162	1449	12.12	1.25
10	12.67	835	1114	14099	#2	943	1258	1449	12.66	1.15
11	14.00	907	1210	15980	#2	1015	1354	1449	13.21	1.07
12	15.33	979	1306	17978	#3	1087	1450	2174	13.77	1.50

Table E14-3.7.2
Summary of Internal Stability Computations for Strength I Load Combinations

E14-3.8 Final MSE Wall Schematic

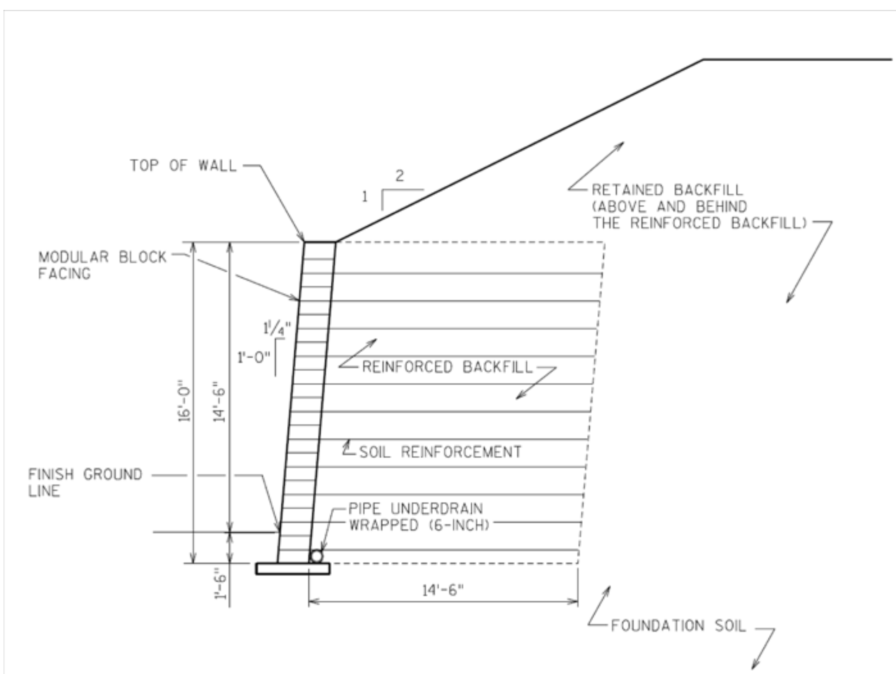

Figure E14-3.8-1
MSE Wall Schematic

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E14-4 Cast-In-Place Concrete Cantilever Wall on Piles, LRFD

General

This example shows design calculations for a cast-in-place (CIP) concrete wall supported on piles conforming to the LRFD Bridge Design Specifications and the WisDOT Bridge Manual. ***(Example is current through LRFD Seventh Edition - 2015 Interim)***

Sample design calculations for pile capacities and wall strength design will be presented. The overall stability and settlement calculations will not be shown in this example, but are required.

Design steps presented in 14.5.2.1 are used for the wall design.

E14-4.1 Establish Project Requirements

The CIP concrete wall shown in Figure E14-4.1-1 will be designed appropriately to accommodate a horizontal backslope. External stability, overall stability and wall strength are the designer's (WisDOT/Consultant) responsibility.

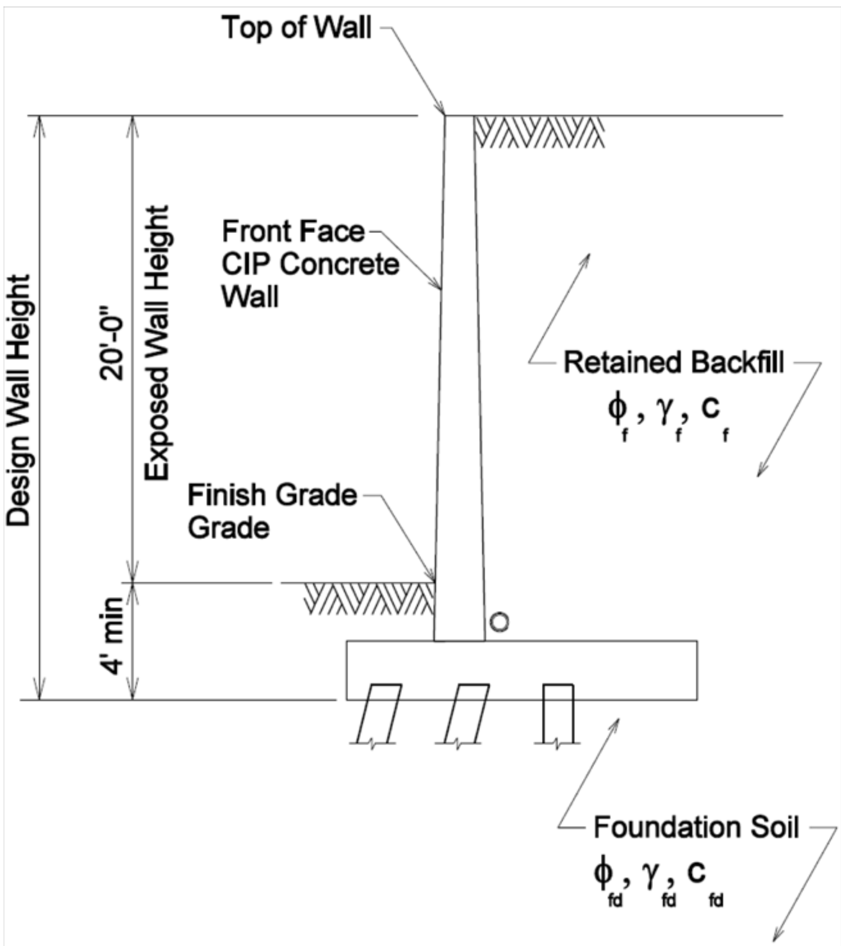

Figure E14-4.1-1
CIP Concrete Wall on Piles

E14-4.2 Design Parameters

Project Parameters

$Design\_Life = 75\ years$ Wall design life (min) **LRFD [11.5.1]**

Soil Properties (From Geotechnical Site Investigation Report)

Designer to determine if long-term or short-term soil strength parameters govern external stability.

Backfill Soil Design Parameters

$\phi_f = 32\ deg$ Angle of internal friction

$\gamma_f = 0.120$ Unit weight, kcf

$c_f = 0$ Cohesion, ksf

$\delta = 17\ deg$ Friction angle between fill and wall

Note: Per WisDOT Bridge Manual and Standard Specifications, structural backfill shall be granular and non-expansive.

ϕ_f= 32 degrees is used for this example, however ϕ_f=30 degrees is the maximum that should be used without testing.

Foundation Soil Design Parameters

$\phi_{fd} = 29\ deg$ Angle of internal friction

$\gamma_{fd} = 0.110$ Unit of weight, kcf

$c_{fd} = 0$ Cohesion, ksf

Reinforced Concrete Parameters

$f'_c = 3.5$ Concrete compressive design strength, ksi (14.5.9)

$\gamma_c = 0.150$ $w_c = \gamma_c$ Unit weight of concrete, ksf

$E_c = 33000\ w_c^{1.5}\sqrt{f'_c}$ Modulus of elasticity of concrete, ksi **LRFD [C5.4.2.4]**

$\boxed{E_c = 3587}$ ksi

$f_y = 60$ Yield strength of reinforcing bars, ksi (14.5.9)

$E_s = 29000$ Modulus of elasticity of reinforcing bars, ksi

Live Load Surcharge Parameters

Live load surcharge shall be used when vehicular load is located within H/2 of the backface of the wall **LRFD [3.11.6.4].** The equivalent height of soil for vehicular load, H_{eq}, used for surcharge loads shall be in accordance to **LRFD [Table 3.11.6.4-2]**. However, WisDOT policy for most cases requires an equivalent height of 2.0 feet. The following procedure is used for determining live load surcharge:

$L_{traffic} = 100.00$ Distance from wall backface to edge of traffic, ft

$\frac{H}{2} = 12.00$ Distance from wall backface where live load surcharge shall be considered in the wall design, ft

Note: The wall height used is the exposed height plus an assumed 4 feet embedment ($H=H_e+4$ feet).

Shall live load surcharge be included? check = "NO"

$h_{eq} = 0.833$ Equivalent height of soil for surcharge load, ft (14.4.5.4.2)

WisDOT Policy: Wall with live load from traffic use 2.0 feet (240 psf) and walls without traffic use 0.833 feet (100 psf)

E14-4.3 Define Wall Geometry

Wall Geometry

$H_e = 20.00$ Exposed wall height, ft

$D_f = 4.00$ Footing cover, ft (WisDOT policy 4'-0" minimum)

$H = H_e + D_f$ Design wall height, ft

$T_t = 1.00$ Stem thickness at top of wall, ft

$b_1 = 0.25$ Front wall batter, in/ft (b_1H:12V)

$b_2 = 0.50$ Back wall batter, in/ft (b_2H:12V)

$\beta = 0.00$ deg Inclination of ground slope behind face of wall, deg (horizontal)

E14-4.7 Evaluate Wall Structural Design

Note: CIP concrete walls are a non-proprietary wall system and the structural design computations shall be performed by the wall designer.

Wall structural design computations for shear and flexure will be considered in this example. Crack control and temperature and shrinkage considerations will also be included.

E14-4.7.1 Evaluate Wall Footing

Investigate shear and moment requirements

E14-4.7.1.1 Evalute One-Way Shear

Design for one-way shear in only the transverse direction.

Compute the effective shear depth,d_v, for the heel:

$cover = 2.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 7$ (transverse bar size)

$Bar_D = 0.875$ in (transverse bar diameter)

$Bar_A = 0.600$ in^2 (transverse bar area)

$$A_{s\_heel} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_heel} = 0.80} \; in^2/ft$$

$$d_{s\_heel} = D\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_heel} = 27.6} \; in$$

$\alpha_1 = 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$$a\_heel = \frac{A_{s\_heel}\, f_y}{\alpha_1\, f'_c\, b} \qquad \boxed{a\_heel = 1.3} \; in$$

$$d_{v1} = d_{s\_heel} - \frac{a\_heel}{2} \qquad \boxed{d_{v1} = 26.9} \; in$$

$$d_{v2} = 0.9\, d_{s\_heel} \qquad \boxed{d_{v2} = 24.8} \; in$$

$$d_{v3} = 0.72\, D\, 12 \qquad \boxed{d_{v3} = 21.6} \; in$$

$$d_{v\_heel} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_heel} = 26.9} \; in$$

Compute the effective shear depth,d_v, for the toe

$cover = 6.0$ in

$s = 9.0$ in (bar spacing)

$Bar_{No} = 7$ (transverse bar size)

$Bar_D = 0.88$ in (transverse bar diameter)

$Bar_A = 0.60$ in^2 (transverse bar area)

$$A_{s\_toe} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_toe} = 0.80} \text{ in}^2\text{/ft}$$

$$d_{s\_toe} = D\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_{s\_toe} = 23.6} \text{ in}$$

$$a\_toe = \frac{A_{s\_toe}\, f_y}{\alpha_1\, f'_c\, b} \qquad \boxed{a\_toe = 1.3} \text{ in}$$

$$d_{v1} = d_{s\_toe} - \frac{a\_toe}{2} \qquad \boxed{d_{v1} = 22.9} \text{ in}$$

$$d_{v2} = 0.9\, d_{s\_toe} \qquad \boxed{d_{v2} = 21.2} \text{ in}$$

$$d_{v\_toe} = \max(d_{v1}, d_{v2}) \qquad \boxed{d_{v\_toe} = 22.9} \text{ in}$$

Determine the distance from Point 'O' to the critical sections:

$$y\_crit\_toe = A\,12 - d_{v\_toe} \qquad \boxed{y\_crit\_toe = 34.1} \text{ in}$$

$$y\_crit\_heel = B\,12 - C\,12 + d_{v\_heel} \qquad \boxed{y\_crit\_heel = 112.0} \text{ in}$$

Determine the distance from Point 'O' to the pile limits:

$$y_{v1\_neg} = y_{p1}\,12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_neg} = 9.1} \text{ in}$$

$$y_{v1\_pos} = y_{p1}\,12 + \frac{B_{yy}}{2} \qquad \boxed{y_{v1\_pos} = 20.9} \text{ in}$$

$$y_{v2\_neg} = y_{p2}\,12 - \frac{B_{yy}}{2} \qquad \boxed{y_{v2\_neg} = 42.1} \text{ in}$$

E14-4.7.1.5 Evaluate Longitudinal Reinforcement Strength

The structural design of the longitudinal reinforcement, assuming the footing acts as a continuous beam over pile supports, is calculated using the maximum pile reactions.

Compute the effective shear depth, d_v ,for the longitudinal reinforcement

$cover = 6.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 5$ (longitudinal bar size)

$Bar_D = 0.625$ in (longitudinal bar diameter)

$Bar_A = 0.310$ in^2 (longitudinal bar area)

$$A_{s\_long} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_long} = 0.31} \ in^2/ft$$

$$d_s = D\,12 - cover - Bar_{D\_toe} - \frac{Bar_D}{2} \qquad \boxed{d_s = 22.8} \ in$$

$$a\_long = \frac{A_{s\_long}\, f_y}{\alpha_1\, f'_c\, b} \qquad \boxed{a\_long = 0.5} \ in$$

$$d_{v1} = d_s - \frac{a\_long}{2} \qquad \boxed{d_{v1} = 22.6} \ in$$

$$d_{v2} = 0.9\, d_s \qquad \boxed{d_{v2} = 20.5} \ in$$

$$d_{v3} = 0.72\, D\, 12 \qquad \boxed{d_{v3} = 21.6} \ in$$

$$d_{v\_long} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_long} = 22.6} \ in$$

Calculate the design moment using a uniform vertical load:

$$L_{pile} = \max(P_1, P_2, P_3) \qquad \boxed{L_{pile} = 8.0} \ ft$$

$$w_u = \frac{V\_Ib}{B} \qquad \boxed{w_u = 3.2} \ kip/ft/ft$$

$$M_u = \frac{w_u\, L_{pile}^2}{10} \qquad \boxed{M_u = 20.3} \ kip\text{-}ft/ft$$

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_{s\_long}\, f_y \left(d_s - \frac{a\_long}{2}\right)\frac{1}{12} \qquad \boxed{M_n = 35.0} \text{ kip-ft/ft}$$

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1} \qquad \boxed{c = 1.58} \text{ in}$$

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases} \qquad \boxed{\phi_F = 0.90}$$

based on f_y = 60 ksi,
LRFD [5.5.4.2.1], [Table C5.7.2.1-1]

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F\, M_n \qquad \boxed{M_r = 31.5} \text{ kip-ft/ft}$$

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.24\sqrt{f'_c} \qquad \boxed{f_r = 0.449} \text{ ksi}$$

$$I_g = \frac{1}{12}\, b\, (D\ 12)^3 \qquad \boxed{I_g = 27000} \text{ in}^4$$

$$y_t = \frac{1}{2}\, D\ 12 \qquad \boxed{y_t = 15.00} \text{ in}$$

$$S_c = \frac{I_g}{y_t} \qquad \boxed{S_c = 1800} \text{ in}^3$$

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12} \qquad \text{from E14-4.7.1.3} \qquad \boxed{M_{cr} = 74.1} \text{ kip-ft/ft}$$

$$\boxed{1.33\, M_u = 27.1} \text{ kip-ft/ft}$$

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

E14-4.7.2 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$H_1 = \gamma_f h_{eq} h' k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_1 = 0.6}$ kip/ft

$H_2 = \frac{1}{2} \gamma_f h'^2 k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_2 = 7.7}$ kip/ft

$M_1 = H_1 \left(\frac{h'}{2}\right)$ $\boxed{M_1 = 6.4}$ kip-ft/ft

$M_2 = H_2 \left(\frac{h'}{3}\right)$ $\boxed{M_2 = 55.2}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$H_{u1} = 1.75\ H_1 + 1.50\ H_2$ $\boxed{H_{u1} = 12.6}$ kip/ft

$M_{u1} = 1.75\ M_1 + 1.50\ M_2$ $\boxed{M_{u1} = 94.0}$ kip-ft/ft

for **Service I**:

$H_{u3} = 1.00\ H_1 + 1.00\ H_2$ $\boxed{H_{u3} = 8.3}$ kip/ft

$M_{u3} = 1.00\ M_1 + 1.00\ M_2$ $\boxed{M_{u3} = 61.6}$ kip-ft/ft

E14-4.7.2.1 Evaluate Stem Shear Strength at Footing

$V_u = H_{u1}$ $\boxed{V_u = 12.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\ \beta \sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 9$ (transverse bar size)

$Bar_D = 1.13$ in (transverse bar diameter)

$Bar_A = 1.00$ in^2 (transverse bar area)

$A_s = \frac{Bar_A}{\frac{s}{12}}$ $\boxed{A_s = 1.00}$ in^2/ft

$d_s = T_b\ 12 - cover - \frac{Bar_D}{2}$ $\boxed{d_s = 25.6}$ in

$a = \frac{A_s\ f_y}{\alpha_1\ f'_c\ b}$ $\boxed{a = 1.7}$ in

$d_{v1} = d_s - \frac{a}{2}$ $\boxed{d_{v1} = 24.7}$ in

$d_{v2} = 0.9\ d_s$ $\boxed{d_{v2} = 23.0}$ in

$d_{v3} = 0.72\ T_b\ 12$ $\boxed{d_{v3} = 20.3}$ in

$d_v = \max(d_{v1}, d_{v2}, d_{v3})$ $\boxed{d_v = 24.7}$ in

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$\beta = 2.0$

$V_c = 0.0316\ \beta\ \sqrt{f'_c}\ b\ d_v$

$V_{n1} = V_c$ $\boxed{V_{n1} = 35.1}$ kip/ft

$V_{n2} = 0.25\ f'_c\ b\ d_v$ $\boxed{V_{n2} = 259.6}$ kip/ft

$V_n = \min(V_{n1}, V_{n2})$ $\boxed{V_n = 35.1}$ kip/ft

$V_r = \phi_v\ V_n$ $\boxed{V_r = 31.6}$ kip/ft

$\boxed{V_u = 12.6}$ kip/ft

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-4.7.2.2 Evaluate Stem Flexural Strength at Footing

$M_u = M_{u1}$ $\boxed{M_u = 94.0}$ kip-ft/ft

Calculate the capacity of the stem in flexure at the face of the footing:

$M_n = A_s\ f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$ $\boxed{M_n = 123.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

E14-4.7.3.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with AASTHO **LRFD [5.10.8]** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 258.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right) f_y}$$

Area of reinforcement per foot, on each face and in each direction $\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \le A_s \le 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

Check the maximum spacing requirements

$s_1 = \min\left(3\ h_s, 18\right)$ $\boxed{s_1 = 18.0}$ in

$$s_2 = \begin{vmatrix} 12 & \text{if} \quad h_s > 18 \\ s_1 & \text{otherwise} \end{vmatrix}$$

For walls and footings (in) $\boxed{s_2 = 18.0}$ in

$s_{max} = \min\left(s_1, s_2\right)$ $\boxed{s_{max} = 18.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-4.8 Summary of Results

List summary of results.

E14-4.8.1 Summary of External Stability

Based on the defined project parameters the following external stability checks have been satisfied:

External Check	CDR
	Strength I
Bearing	1.46
Eccentricity	> 10
Sliding	1.12

Table E14-4.8-1
Summary of External Stability Computations

E14-4.8.2 Summary of Wall Strength Design

The required wall reinforcing from the previous computations are presented in Figure E14-6.9-1.

E14-4.8.3 Drainage Design

Drainage requirements shall be investigated and detailed accordingly. In this example drainage requirements are met by providing granular, free draining backfill material with a pipe underdrain located at the bottom of the wall (Assumed wall is adjacent to sidewalk) as shown in Figure E14-4.9-1.

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WisDOT exception to AASHTO:

A 3-inch minimum panel thickness is used, even though **LRFD [9.7.4.3.1]** specifies a minimum thickness of 3.5 inches.

The decision to use a 3-inch minimum panel was based on the successful use of 3-inch panels by other agencies over many years. In addition, a minimum of 5 inches of cast-in-place concrete is preferred for crack control and reinforcing steel placement. A 3.5-inch panel thickness would require an 8.5-inch deck, which would not allow direct substitution of panels for a traditionally designed 8-inch deck.

A study performed at Iowa State University determined that a 3-inch thick panel with coated 3/8-inch strands at midthickness spaced at 6 inches, along with epoxy-coated 6 x 6 – D6 x D6 welded wire fabric, was adequate to prevent concrete splitting during strand detensioning. The use of #3 bars placed perpendicular to the strands at 9" spacing also prevents concrete splitting.

Panel thicknesses were increased by ½ inch whenever a strand spacing of less than 6 inches was required. Strands with a ½-inch diameter were used in panels 3½ inches thick or greater when 3/8-inch strands spaced at 6 inches were not sufficient.

The allowable tensile stress in the panels, as presented in **LRFD [Table 5.9.4.2.2-1]**, is as follows:

$$0.0948\sqrt{f'_c} \leq 0.3 \text{ ksi}$$

This allowable tensile stress limit is based on f'_c in units of ksi and is for components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions.

The transfer length of the strands is assumed to be 60 strand diameters at a stress of 202.5 ksi. The development length, L_d, of the strands, as presented in **LRFD [5.11.4.2]**, is assumed to be as follows:

$$L_d = k\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b$$

Where:

k	=	1.0 for pretensioned members with a depth less than 24 inches
d_b	=	Nominal strand diameter (inches)
f_{ps}	=	Average stress in prestressing steel at the time when the nominal resistance of the member is required (ksi)

f_{pe} = Effective stress in prestressing steel after losses (ksi)

L_d = Development length beyond critical section (inches)

The minimum panel width is the length required for the panel to extend 4" onto the top flange as shown in Table 17.10-1. A linear reduction in f_{pe} is required if the panel width is less than two times the development length. The values shown in Table 17.10-1 consider this linear reduction.

The designs in Table 17.10-1 are based on uncoated prestressing strands. Grit-impregnated, epoxy-coated strands cost four times as much as uncoated strands but require about half the transfer and development length as uncoated strands. A cover of 1 1/4 inches is adequate to provide protection from chlorides for uncoated strands using a 5 ksi concrete mix. However, for bridges with high traffic volume, a 6 ksi mix is recommended.

LRFD [9.7.4.3.2] specifies that the strands need not extend beyond the panels into the cast-in-place concrete above the beams. This simplifies construction of the panels at the plant since they can be saw cut to the required length. Installation in the field is also simplified because extended strands may interfere with girder shear connectors. As a substitute for the strands that don't extend out of the panels, #4 bars spaced at twice the spacing of the transverse bars are placed on top of the panels over the girders in the cast-in-place concrete. These bars anchor the panels together to prevent or reduce longitudinal cracking over the ends of the panels and also resist any positive continuity moments that may develop. Also by not extending the strands into the cast-in-place concrete, the uncoated strands are not exposed to chlorides that may seep through cracks that may develop in the cast-in-place concrete.

LRFD [5.7.3.3.2] requires that the moment capacity of a flexural member be greater than the cracking moment based on the modulus of rupture. This requirement may be waived if the moment capacity is greater than 1.33 times the factored design moment. The purpose of this requirement is to provide a minimum amount of reinforcement in a flexural member so that a flexural failure will not be sudden or occur without warning. Tests have shown that for slabs on girders, the failure mode is a punching shear failure and not a flexural failure. ACI 10.5.4 also recognizes the difference between slabs and beams and does not require the same minimum reinforcement for slabs. For these reasons, **LRFD [5.7.3.3.2]** was not considered in the designs of the panels shown in Table 17.10-1. However, panels with a width of 6 feet or more meet the requirements of **LRFD [5.7.3.3.2]**.

17.10.3 Transverse Reinforcement for Cast-in-Place Concrete on Deck Panels

The design of the transverse reinforcing steel in the cast-in-place concrete placed on deck panels is based on *AASHTO LRFD*. The live load moments used to determine the size and spacing of the transverse reinforcing bars placed in the top of the cast-in-place concrete are from **LRFD [Table A4-1]**. The reinforcing steel in the cast-in-place concrete is also designed for a future wearing surface of 20 psf. With stay-in-place forms, there are no negative moments from the dead load of the cast-in-place concrete. The required reinforcing steel shown in Table 17.10-2 is based on both the strength requirement and crack control requirement.

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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.

18.2 Specifications, Material Properties and Structure Type

18.2.1 Specifications

Reference may be made to the design and construction related material as presented in the following specifications:

- *State of Wisconsin, Department of Transportation Standard Specifications for Highway and Structure Construction*

  Section 502 - Concrete Bridges

  Section 505 - Steel Reinforcement

- Other Specifications as referenced in Chapter 3

18.2.2 Material Properties

The properties of materials used for concrete slab structures are as follows:

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

4 ksi, for concrete slab superstructure

3.5 ksi, for concrete substructure units

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in slab **LRFD [C5.4.2.4]**

= $33{,}000\ K_1\ w_c^{1.5}\ (f'_c)^{1/2} = 3800$ ksi

Where:

K_1 = 1.0

w_c = 0.150 kcf, unit weight of concrete

n = $E_s / E_c = 8$ **LRFD [5.7.1]** (modular ratio)

18.2.3 Structure Type and Slab Depth

Prepare preliminary structure data, looking at the type of structure, span lengths, approximate slab depth, skew, roadway width, etc.. The selection of the type of concrete slab

- Set value of load modifier, η_i , and its factors (η_D , η_R , η_I) all equal to 1.00 for concrete slab design.
- Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects, Q_i , for slab bridges.
- Ignore reduction factor, r, for skewed slab bridges in **LRFD [4.6.2.3]** that would reduce longitudinal force effects, Q_i .

18.3.3 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a bridge is expected to experience during its design life **LRFD [1.3.2.4]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Strength I Limit State **LRFD [3.4.1]** will be used for:

- Designing longitudinal slab reinforcement for flexure
- Designing transverse slab reinforcement over the piers for flexure
- Checking shear (two-way) in slab at the piers
- Checking uplift at the abutments
- Checking longitudinal slab reinforcement for tension from shear

18.3.3.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Strength I Limit State will be used to design the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, defined in 18.4.3.2.

The load factor, γ_i , is used to adjust force effects on a structural element. This factor accounts for variability of loads, lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

For Strength I Limit State, the values of γ_i for each applied load, are found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]** and their values are: γ_{DC} = 1.25/0.90, γ_{DW} = 1.50/0.65, 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 [5.7.2.1]**)

- ϕ = 0.90 for shear and torsion

The factored resistance, R_r (M_r , V_r , T_{cap}), associated with the list of items to be designed/checked using Strength I Limit State in 18.3.3, are described in the following sections.

18.3.3.2.1 Moment Capacity

Stress is assumed proportional to strain below the proportional limit on the stress-strain diagram. Tests have shown that at high levels of stress in concrete, stress is not proportional to strain. Recognizing this fact, strength analysis takes into account the nonlinearity of the stress-strain diagram. This is accomplished by using a rectangular stress block to relate the concrete compressive stress distribution to the concrete strain. The compressive stress block has a uniform value of $\alpha_1 \cdot f'_C$ over a zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance a = $\beta_1 \cdot$(c) from the extreme compression fiber. The distance (c) is measured perpendicular to the neutral axis. The factor α_1 shall be taken as 0.85 for concrete strengths not exceeding 10.0 ksi and the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi **LRFD [5.7.2.2]**. Strength predictions using this method are in agreement with strength test results. The representation of these assumptions is shown in Figure 18.3-1.

The moment capacity (factored resistance) of concrete components shall be based on the conditions of equilibrium and strain compatibility, resistance factors as specified in **LRFD [5.5.4.2]** and the assumptions outlined in **LRFD [5.7.2]**.

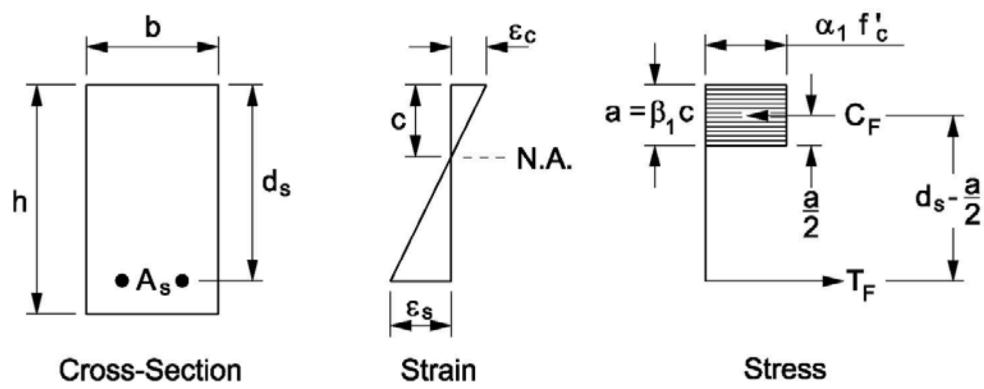

Figure 18.3-1
Stress / Strain on Cross - Section

Referring to Figure 18.3-1, the internal force equations are:

$C_F = \alpha_1 \cdot (f'_c)\ (b)\ (a) = 0.85\ (f'_c)\ (b)\ (a)$

$T_F = (A_s)\ (f_s)$

By equating C_F to T_F, and solving for the compressive stress block depth, (a), gives:

$a = A_s f_s / 0.85\ (f'_c)\ (b)$

Use ($f_s = f_y$) when the steel yields prior to crushing of the concrete. To check for yielding, assume ($f_s = f_y$) and calculate the value for (a). Then calculate the value for $c = a / \beta_1$ and d_s as shown in Figure 18.3-1. If c / d_s does not exceed the value calculated below, then the reinforcement has yielded and the assumption is correct, as stated in **LRFD [5.7.2.1]**.

$c / d_s \leq 0.003 / (0.003 + \varepsilon_{cl})$

ε_{cl} = compression controlled strain limit

for f_y = 60 ksi, ε_{cl} is 0.0020 per **LRFD[Table C5.7.2.1-1]**

if $c / d_s \leq 0.6$, then the reinforcement (f_y = 60 ksi) will yield and ($f_s = f_y$)

For rectangular sections, the nominal moment resistance, M_n , (tension reinforcement only) equals: **LRFD [5.7.3.2.3]**

$M_n = A_s f_s (d_s - a/2)$

The factored resistance, M_r , or moment capacity, shall be taken as: **LRFD [5.7.3.2.1]**

$M_r = \phi M_n = \phi A_s f_s (d_s - a/2)$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

$$M_r = (0.9)\ A_s\ f_s\ (d_s – a/2)$$

18.3.3.2.2 Shear Capacity

The nominal shear resistance, V_n , for two-way action, shall be determined as: **LRFD [5.8.1.4, 5.13.3.6.3]**

$$V_n = (0.063 + 0.126 / \beta_c)\ (f'_c)^{½}\ b_o\ d_v \leq 0.126\ (f'_c)^{½}\ b_o\ d_v \quad \text{(kips)}$$

Where:

f'_c	=	4.0 ksi (for concrete slab bridges)
β_c	=	ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted
d_v	=	effective shear depth as determined in **LRFD [5.8.2.9]** (in)
b_o	=	perimeter of the critical section (in)

The factored resistance, V_r , or shear capacity, shall be taken as: **LRFD [5.8.2.1]**

$$V_r = \phi\ V_n$$

The resistance factor, ϕ, is 0.90, therefore:

$$V_r = (0.9)\ V_n$$

18.3.3.2.3 Uplift Check

The check of uplift at abutments does not use a factored resistance, but compares factored dead load and live load reactions.

18.3.3.2.4 Tensile Capacity – Longitudinal Reinforcement

The nominal tensile resistance, T_{nom} , for an area, A_s , of developed reinforcement, equals:

$$T_{nom} = A_s\ f_y$$

The factored resistance, T_{cap} , or tensile capacity, shall be taken as:

$$T_{cap} = \phi\ T_{nom} = \phi\ A_s\ f_y$$

For tension-controlled reinforced concrete sections, the resistance factor, ϕ, is 0.90, therefore:

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E18-1 Continuous 3-Span Haunched Slab - LRFD

A continuous 3-span haunched slab structure is used for the design example. The same basic procedure is applicable to continuous flat slabs. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. Design using a slab width equal to one foot. (***Example is current through LRFD Seventh Edition - 2015 Interim***)

E18-1.1 Structure Preliminary Data

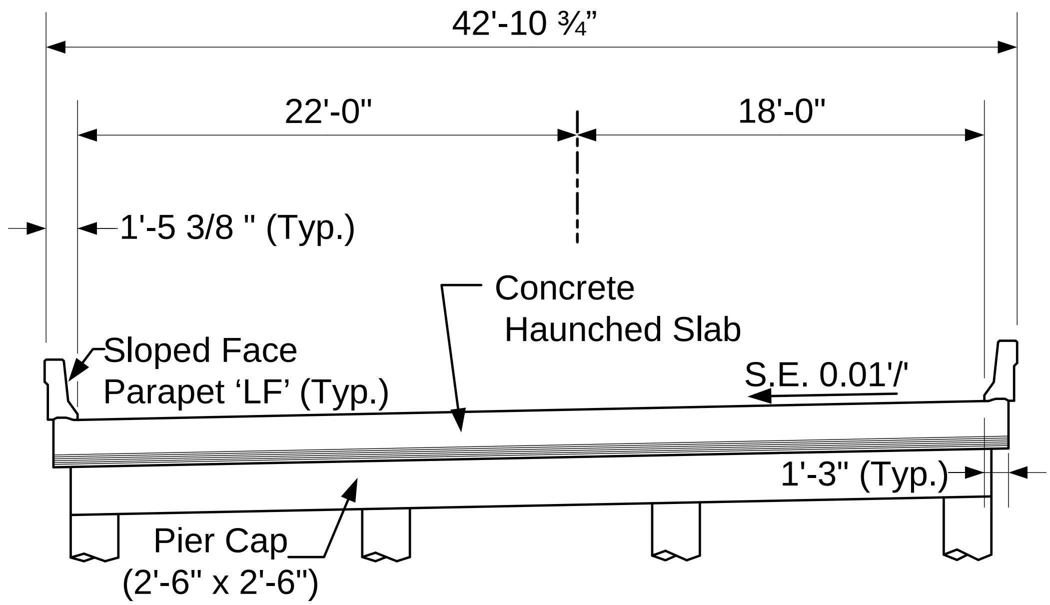

Figure E18.1

Section Perpendicular to Centerline

Live Load: HL-93
(A1) Fixed Abutments at both ends
Parapets placed after falsework is released

Geometry:

$L_1 := 38.0$	ft	Span 1
$L_2 := 51.0$	ft	Span 2
$L_3 := 38.0$	ft	Span 3
$slab_{width} := 42.5$	ft	out to out width of slab
$skew := 6$	deg	skew angle (RHF)
$w_{roadway} := 40.0$	ft	clear roadway width

Material Properties: (See 18.2.2)

$f'_c := 4$	ksi	concrete compressive strength

$f_y := 60$ ksi yield strength of reinforcement

$E_c := 3800$ ksi modulus of elasticity of concrete

$E_s := 29000$ ksi modulus of elasticity of reinforcement

$n := 8$ E_s / E_c (modular ratio)

Weights:

$w_c := 150$ pcf concrete unit weight

$w_{LF} := 387$ plf weight of Type LF parapet (each)

E18-1.2 LRFD Requirements

For concrete slab design, the slab dimensions and the size and spacing of reinforcement shall be selected to satisfy the equation below for all appropriate Limit States: (See 18.3.2.1)

$$Q = \Sigma\eta_i \cdot \gamma_i \cdot Q_i \le \phi \cdot R_n = R_r \qquad \text{(Limit States Equation)}$$

The value of the load modifier is:

$\eta_i := 1.0$ for all Limit States (See 18.3.2.2)

The force effect, Q_i , is the moment, shear, stress range or deformation caused by applied loads.

The applied loads from **LRFD [3.3.2]** are:

DC = dead load of slab (DC_{slab}), ½ inch wearing surface ($DC_{1/2"WS}$) and parapet dead load (DC_{para}) - (See E18-1.3)

DW = dead load of future wearing surface (DW_{FWS}) - (See E18-1.3)

LL+IM = vehicular live load (LL) with dynamic load allowance (IM) - (See E18-1.4)

The Influence of ADTT and skew on force effects, Q_i , are ignored for slab bridges (See 18.3.2.2).

The values for the load factors, γ_i , (for each applied load) and the resistance factors, ϕ , are found in Table E18.1.

The total factored force effect, Q , must not exceed the factored resistance, R_r . The nominal resistance, R_n , is the resistance of a component to the force effects.

In Table E18.4:

[1] M_{DC} is moment due to slab dead load (DC_{slab}), parapet dead load (DC_{para}) after its weight is distributed across width of slab, and 1/2 inch wearing surface ($DC_{1/2"WS}$).

[2] M_{DW} is moment due to future wearing surface (DW_{FWS}).

[3] The points of contraflexure are located at the (0.66 pt.) of span 1 and the (0.25 pt.) of span 2, when a uniform load is placed across the entire structure. Negative moments in these columns are shown between the points of contraflexure per **LRFD [3.6.1.3.1]**.

E18-1.7 Longitudinal Slab Reinforcement (Interior Strip)

Select longitudinal reinforcement for an Interior Strip.

The concrete cover on the top bars is 2 1/2 inches, which includes a 1/2 inch wearing surface. The bottom bar cover is 1 1/2 inches. (See 18.4.6)

E18-1.7.1 Positive Moment Reinforcement for Span 1

Examine the 0.4 point of span 1

E18-1.7.1.1 Design for Strength

Design reinforcement using Strength I Limit State and considerations and assumptions detailed in **LRFD [5.5.4, 5.7.2]**

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DCmax} := 1.25$ $\gamma_{DWmax} := 1.50$ $\gamma_{LLstr1} := 1.75$ $\phi_f := 0.9$

$Q_i = M_{DC}, M_{DW}, M_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; moments due to applied loads as stated in E18-1.2

$$Q = M_u = \eta_i\,[\gamma_{DCmax}(M_{DC}) + \gamma_{DWmax}(M_{DW}) + \gamma_{LLstr1}(M_{LL+IM})]$$

$$= 1.0\,[1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})]$$

$$R_n = M_n = A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right)$$ (See 18.3.3.2.1)

$$M_r = \phi_f \cdot M_n = 0.90 \cdot A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right)$$

Therefore : $M_u \le M_r$ (Limit States Equation)

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \le 0.90\,A_s\,f_s\,(d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM)

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.4 pt. - span 1):

M_{DC} = 18.1 kip-ft M_{DW} = 1.5 kip-ft M_{LL+IM} = 7.9 + 37.5 = 45.4 kip-ft

$M_u := 1.25\cdot(18.1) + 1.50\cdot(1.5) + 1.75\cdot(45.4)$ $\boxed{M_u = 104.3}$ kip-ft

$b := 12$ inches (for a one foot design width)

$d_s = d_{slab}$ - bott. bar clr. - 1/2 bott. bar dia.

$d_s := 17 - 1.5 - 0.6$ $\boxed{d_s = 14.9}$ in

Calculate R_u, coefficient of resistance:

$R_u = \dfrac{M_u}{\phi_f \cdot b \cdot d_s^{\,2}}$ $R_u := \dfrac{104.3\cdot(12)\cdot 1000}{0.9\cdot(12)\cdot 14.9^2}$ $\boxed{R_u = 522}$ psi

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.0095$

$A_s = \rho\cdot(b)\cdot d_s$ $A_s := 0.0095\cdot(12)14.9$ $\boxed{A_s = 1.7}$ $\dfrac{in^2}{ft}$

Try: #9 at 7" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

Calculate the depth of the compressive stress block.

Assume $f_s = f_y$ (See 18.3.3.2.1) ; for f'_c = 4.0 ksi : $\alpha_1 := 0.85$ and $\beta_1 = 0.85$

$a = \dfrac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot b}$ $a := \dfrac{1.71\cdot(60)}{0.85\cdot(4.0)\cdot 12}$ $\boxed{a = 2.51}$ in

If $\dfrac{c}{d_s} \le 0.6$ for (f_y= 60 ksi) **LRFD [5.7.2.1]**, then reinforcement has yielded and the assumption is correct.

$\beta_1 := 0.85$ $c := \dfrac{a}{\beta_1}$ $\boxed{c = 2.96}$ in

$\dfrac{c}{d_s} = 0.2 < 0.6$ therefore, the reinforcement will yield.

$M_r = 0.90\cdot A_s \cdot f_y \cdot \left(d_s - \dfrac{a}{2}\right)$

$M_r := 0.9\cdot(1.71)\cdot 60.0\cdot\left(\dfrac{14.9 - \dfrac{2.51}{2}}{12}\right)$ $\boxed{M_r = 105}$ kip-ft

$$V_r = \phi_v \cdot V_n = \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_v) \le \phi_v \cdot (0.126) \cdot \sqrt{f'_c} \cdot (b_o) \cdot (d_v)$$

Where:

β_c = ratio of long side to short side of the rectangle through which reaction force is transmitted
$\approx$ 41.71 ft. / 2.5 ft. = 16.7

d_v = effective shear depth = dist. between resultant tensile & compressive forces
$\approx$ 24 in.

b_o = perimeter of the critical section
$\approx$ 1109 in.

Therefore, $V_r := \phi_v \cdot \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v$ $\boxed{V_r = 3380}$ kips

but $\le \phi_v \cdot 0.126 \cdot \sqrt{f'_c} \cdot (b_o) \cdot d_v = 6036$ kips

Therefore, V_u = 1336 kips < V_r = 3380 kips O.K.

Note: Shear check and shear reinforcement design for the pier cap is not shown in this example. Also crack control criteria, minimum reinforcement checks, and shrinkage and temperature reinforcement checks are not shown for the pier cap.

E18-1.16.8 Minimum Reinforcement Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column) for minimum reinforcement criteria.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD [5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

from E18-1.7.1.4, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$f_r = 0.24\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.24\sqrt{4}$ $\boxed{f_r = 0.48}$ ksi

h = pier cap depth + D_{haunch} (section depth) $\boxed{h = 58}$ in

b_{cap} = pier cap width $\boxed{b_{cap} = 30}$ in

$I_g := \frac{1}{12} \cdot b_{cap} \cdot h^3$ (gross moment of inertia) $\boxed{I_g = 487780}$ in^4

$c := \frac{h}{2}$ (section depth/2) $\boxed{c = 29}$ in

$$M_{cr} = \frac{1.1 f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.48 \cdot (487780)}{29(12)} \qquad \boxed{M_{cr} = 740.1} \text{ kip-ft}$$

$1.33 \cdot M_u = 605.4$ kip-ft , where M_u was calculated for Strength Design in E18-1.16.6.1 and (M_u = 455.2 kip-ft)

1.33 M_u controls because it is less than M_{cr}

Recalculating requirements for (New moment = $1.33 \cdot M_u$ = 605.4 kip-ft)

$\boxed{b_{neg} = 30}$ in (See E18-1.16.2)

$\boxed{d_{neg} = 54.62}$ in (See E18-1.16.2)

Calculate R_u, coefficient of resistance:

$$R_u = \frac{M_u}{\phi_f \cdot (b_{neg}) \cdot d_{neg}^2} \qquad R_u := \frac{605.4 \cdot (12) \cdot 1000}{0.9(30) \cdot 54.62^2} \qquad \boxed{R_u = 90.2} \text{ psi}$$

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.00152$

$A_s = \rho \cdot (b_{neg}) \cdot d_{neg}$ $\quad A_s := 0.00152 \cdot (30) 54.62$ $\quad \boxed{A_s = 2.49}$ in^2

Place this reinforcement in a width, centered over the pier, equal to 1/2 the center to center column spacing or 8 feet, whichever is smaller. Therefore, width equals 6.5 feet.

Therefore, 2.49 in^2/6.5 ft. = 0.38 in^2/ft. Try #5 at 9" c-c spacing for a 6.5 ft. transverse width over the pier. This will provide (A_s = 2.79 in^2) in a 6.5 ft. width.

Calculate the depth of the compressive stress block

Assume $f_s = f_y$ (See 18.3.3.2.1) ; for f'_c = 4.0 ksi : $\alpha_1 := 0.85$ and $\beta_1 = 0.85$

$$a = \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot b_{neg}} \qquad a := \frac{2.79 \cdot (60)}{0.85 \cdot (4.0) \cdot 30} \qquad \boxed{a = 1.64} \text{ in}$$

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[5.4.4.2]

E_{ct} = Modulus of elasticity of concrete at transfer or time of load application in ksi (see 19.3.3.8)

f_{gcp} = Concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)

19.3.2.2.2 Time-Dependent Losses

Per **LRFD [5.9.5.3],** an estimate of the long-term losses due to steel relaxation as well as concrete creep and shrinkage on standard precast, pretensioned members shall be taken as:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

Where:

$$\gamma_h = 1.7 - 0.01H$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})}$$

f_{pi} = Prestressing steel stress immediately prior to transfer (ksi)

H = Average annual ambient relative humidity in %, taken as 72% in Wisconsin

Δf_{pR} = Relaxation loss estimate taken as 2.4 ksi for low relaxation strands or 10.0 ksi for stress-relieved strands (ksi)

The losses due to elastic shortening must then be added to these time-dependent losses to determine the total losses. For members made without composite deck slabs such as box girders, time-dependent losses shall be determined using the refined method of **LRFD [5.9.5.4]**. For non-standard members with unusual dimensions or built using staged segmental construction, the refined method of **LRFD [5.9.5.4]** shall also be used.

19.3.2.2.3 Fabrication Losses

Fabrication losses are not considered by the designer, but they affect the design criteria used during design. Anchorage losses which occur during stressing and seating of the prestressed strands vary between 1% and 4%. Losses due to temperature change in the strands during cold weather prestressing are 6% for a 60°F change. The construction specifications permit a 5% difference in the jack pressure and elongation measurement without any adjustment.

19.3.2.3 Service Load

During service load, the member is subjected to the same loads that are present after prestress transfer and losses occur, in addition to the effects of the I-girder and box girder load-carrying behavior described in the next two sections.

19.3.2.3.1 I-Girder

In the case of an I-girder, the dead load of the deck and diaphragms are always carried by the basic girder section on a simple span. At strand release, the girder dead load moments are calculated based on the full girder length. For all other loading stages, the girder dead load moments are based on the span length. This is due to the type of construction used (that is, nonshored girders simply spanning from one substructure unit to another for single-span as well as multi-span structures).

The live load plus dynamic load allowance along with any superimposed dead load (curb, parapet or median strip which is placed after the deck concrete has hardened) are carried by the continuous composite section.

WisDOT exception to AASHTO:

The standard pier diaphragm is considered to satisfy the requirements of **LRFD [5.14.1.4.5]** and shall be considered to be fully effective.

In the case of multi-span structures with fully effective diaphragms, the longitudinal distribution of the live load, dynamic load allowance and superimposed dead loads are based on a continuous span structure. This continuity is achieved by:

a. Placing non-prestressed (conventional) reinforcement in the deck area over the interior supports.

b. Casting concrete between and around the abutting ends of adjacent girders to form a diaphragm at the support. Girders shall be in line at interior supports and equal numbers of girders shall be used in adjacent spans. The use of variable numbers of girders between spans requires prior approval by BOS.

If the span length ratio of two adjacent spans exceeds 1.5, the girders are designed as simple spans. In either case, the stirrup spacing is detailed the same as for continuous spans and bar steel is placed over the supports equivalent to continuous span design. It should be noted that this value of 1.5 is not an absolute structural limit.

19.3.2.3.2 Box Girder

In the case of slabs and box girders with a bituminous or thin concrete surface, the dead load together with the live load and dynamic load allowance are carried by the basic girder section.

When this girder type has a concrete floor, the dead load of the floor is carried by the basic section and the live load, dynamic load allowance and any superimposed dead loads are

19.3.3.4 Live Load

The HL-93 live load shall be used for all new bridges. Refer to section 17.2.4.2 for a detailed description of the HL-93 live load, including the design truck, design tandem, design lane, and double truck.

19.3.3.5 Live Load Distribution

The live load distribution factors shall be computed as specified in **LRFD [4.6.2.2]** and as summarized in Table 17.2-7. The moment and shear distribution factors are determined using equations that consider girder spacing, span length, deck thickness, the number of girders, skew and the longitudinal stiffness parameter. Separate shear and moment distribution factors are computed for interior and exterior girders. The applicability ranges of the distribution factors shall also be considered. If the applicability ranges are not satisfied, then conservative assumptions must be made based on sound engineering judgment.

WisDOT policy item:

The typical cross section for prestressed adjacent box girders shall be type "g" as illustrated in **LRFD [Table 4.6.2.2.1-1]**. The connection between the adjacent box girders shall be considered to be only enough to prevent relative vertical displacement at the interface.

The St. Venant torsional inertia, J, for adjacent box beams with voids may be calculated as specified for closed thin-walled sections in accordance with **LRFD [C4.6.2.2.1]**.

The value of poisson's ratio shall be taken as 0.2 in accordance with **LRFD [5.4.2.5]**.

The beam spacing, S, in **LRFD [Table 4.6.2.2b-1]** shall be equal to the beam width plus the space between adjacent box sections.

See 17.2.8 for additional information regarding live load distribution.

19.3.3.6 Dynamic Load Allowance

The dynamic load allowance, IM, is given by **LRFD [3.6.2]**. Dynamic load allowance equals 33% for all live load limit states except the fatigue limit state and is not applied to pedestrian loads or the lane load portion of the HL-93 live load. See 17.2.4.3 for further information regarding dynamic load allowance.

19.3.3.7 Deck Design

The design of concrete decks on prestressed concrete girders is based on **LRFD [4.6.2.1]**. Moments from truck wheel loads are distributed over a width of deck which spans perpendicular to the girders. This width is known as the distribution width and is given by **LRFD [Table 4.6.2.1.3-1]**. See 17.5 for further information regarding deck design.

19.3.3.8 Composite Section

The effective flange width is the width of the deck slab that is to be taken as effective in composite action for determining resistance for all limit states. The effective flange width, in accordance with **LRFD [4.6.2.6]**, is equal to the tributary width of the girder for interior girders. For exterior girders, it is equal to one half the effective flange width of the adjacent interior girder plus the overhang width. The effective flange width shall be determined for both interior and exterior beams.

For box beams, the composite flange area for an interior multi-beam is taken as the width of the member by the effective thickness of the floor. Minimum concrete overlay thickness is 3”. The composite flange for the exterior member consists of the curb and the floor over that particular edge beam. Additional information on box girders may be found in 17.4.

Since the deck concrete has a lower strength than the girder concrete, it also has a lower modulus of elasticity. Therefore, when computing composite section properties, the effective flange width (as stated above) must be reduced by the ratio of the modulus of elasticity of the deck concrete divided by the modulus of elasticity of the girder concrete.

WisDOT exception to AASHTO:

WisDOT uses the formulas shown below to determine E_c for prestressed girder design. For 6 ksi girder concrete, E_c is 5,500 ksi, and for 4 ksi deck concrete, E_c is 4,125 ksi. The E_c value of 5,500 ksi for 6 ksi girder concrete strength was determined from deflection studies. These equations are used in place of those presented in **LRFD [5.4.2.4]** for the following calculations: strength, section properties, and deflections due to externally applied dead and live loads.

For slab concrete strength other than 4 ksi, E_c is calculated from the following formula:

$$E_c = \frac{4{,}125\sqrt{f'_c}}{\sqrt{4}} \text{ (ksi)}$$

For girder concrete strengths other than 6 ksi, E_c is calculated from the following formula:

$$E_c = \frac{5{,}500\sqrt{f'_c}}{\sqrt{6}} \text{ (ksi)}$$

WisDOT policy item:

WisDOT uses the equation presented in **LRFD [C5.4.2.4]** (and shown below) to calculate the modulus of elasticity at the time of release using the specified value of f’ci. This value of E_i is used for loss calculations and for girder camber due to prestress forces and girder self weight.

$$E_c = 33{,}000 \cdot K_1 \cdot w_c^{1.5}\sqrt{f'_{ci}}$$

Where:

K_1	=	Correction factor for source of aggregate, use 1.0 unless previously approved by BOS.
w_c	=	Unit weight of concrete, 0.150 (kcf)
f'_{ci}	=	Specified compressive strength of concrete at the time of release (ksi)

19.3.3.9 Design Stress

In many cases, stress at the Service III limit state in the bottom fiber at or near midspan after losses will control the flexural design. Determine a trial strand pattern for this condition and proceed with the flexural design, adjusting the strand pattern if necessary.

The design stress is the sum of the Service III limit state bottom fiber stresses due to non-composite dead load on the basic girder section, plus live load, dynamic load allowance and superimposed dead load on the composite section, as follows:

$$f_{des} = \frac{M_{d(nc)}}{S_{b(nc)}} + \frac{M_{d(c)} + M_{(LL+IM)}}{S_{b(c)}}$$

Where:

f_{des}	=	Service III design stress at section (ksi)
$M_{d(nc)}$	=	Service III non-composite dead load moment at section (k-in)
$M_{d(c)}$	=	Service III superimposed dead load moment at section (k-in)
$M_{(LL+IM)}$	=	Service III live load plus dynamic load allowance moment at section (k-in)
$S_{b(nc)}$	=	Non-composite section modulus for bottom of basic beam (in^3)
$S_{b(c)}$	=	Composite section modulus for bottom of basic beam (in^3)

The point of maximum stress is generally 0.5 of the span for both end and intermediate spans. But for longer spans (over 100'), the 0.4 point of the end span may control and should be checked.

19.3.3.10 Prestress Force

With f_{des} known, compute the required effective stress in the prestressing steel after losses, f_{pe}, needed to counteract all the design stress except an amount of tension equal to the tensile stress limit listed in **LRFD [Table 5.9.4.2.2-1]**. The top of the girder is subjected to severe corrosion conditions and the bottom of the girder is subjected to moderate exposure. The Service III tensile stress at the bottom fiber after losses for pretensioned concrete shall not exceed $0.19\sqrt{f'_c}$ (or 0.6 ksi). Therefore:

$$f_{pe} = f_{des} - \min(0.19\sqrt{f'_c} \text{ or } 0.6 \text{ ksi})$$

Note: A conservative approach used in hand calculations is to assume that the allowable tensile stress equals zero.

Applying the theory discussed in 19.2:

$$f_{pe} = \frac{P_{pe}}{A}\left(1 + \frac{ey}{r^2}\right)$$

Where:

P_{pe} = Effective prestress force after losses (kips)

A = Basic beam area (in^2)

e = Eccentricity of prestressing strands with respect to the centroid of the basic beam at section (in)

r = $\sqrt{\frac{I}{A}}$ of the basic beam (in)

For slab and box girders, assume an e and apply this to the above equation to determine P_{pe} and the approximate number of strands. Then a trial strand pattern is established using the Standard Details as a guide, and a check is made on the assumed eccentricity. For I-girders, f_{pe} is solved for several predetermined patterns and is tabulated in the Standard Details.

Present practice is to detail all spans of equal length with the same number of strands, unless a span requires more than three additional strands. In this case, the different strand arrangements are detailed along with a plan note stating: "The manufacturer may furnish all girders with the greater number of strands."

19.3.3.11 Service Limit State

Several checks need to be performed at the service limit state. Refer to the previous narrative in 19.3.3 for sections to be investigated and section 17.2.3.2 for discussion on the service limit state. Note that Service I limit state is used when checking compressive stresses and Service III limit state is used when checking tensile stresses.

The following should be verified by the engineer:

- Verify that the Service III tensile stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed the limits presented in **LRFD [Table 5.9.4.1.2-1]**, which depend upon whether or not the strands are bonded and satisfy stress requirements. This will generally control at the top of the beam near the beam ends where the dead load moment approaches zero and is not able to counter the tensile stress at the top of the beam induced by the prestress force. When the calculated tensile stress exceeds the stress limits, the strand pattern must be modified by draping or partially debonding the strand configuration.

- Verify that the Service I compressive stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed 0.60 f'_{ci}, as presented in **LRFD [5.9.4.1.1]**. This will generally control at the bottom of the beam near the beam ends or at the hold-down point if using draped strands.

- Verify that the Service III tensile stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.4.2.2-1]**. No tensile stress shall be permitted for unbonded strands. The tensile stress of bonded strands shall not exceed $0.19\sqrt{f'_c}$ (or 0.6 ksi) as all strands shall be considered to be in moderate corrosive conditions. This will generally control at the bottom of the beam near midspan and at the top of the continuous end of the beam.

- Verify that the Service I compressive stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.4.2.1-1]**. Two checks need to be made for girder bridges. The compressive stress due to the sum of effective prestress and permanent loads shall not exceed 0.45 f'_c (ksi). The compressive stress due to the sum of effective prestress, permanent loads and transient loads shall not exceed $0.60\phi_w f'_c$ (ksi). The term ϕ_w, a reduction factor applied to thin-walled box girders, shall be 1.0 for WisDOT standard girders.

- Verify that Fatigue I compressive stress due to fatigue live load and one-half the sum of effective prestress and permanent loads does not exceed 0.40 f'_c (ksi) **LRFD [5.5.3.1]**.

- Verify that the Service I compressive stress at the top of the deck due to all dead and live loads applied to the appropriate sections after losses does not exceed 0.40 f'_c.

WisDOT policy item:

The top of the prestressed girders at interior supports shall be designed as reinforced concrete members at the strength limit state in accordance with **LRFD [5.14.1.4.6]**. In this case, the stress limits for the service limit state shall not apply to this region of the precast girder.

19.3.3.12 Raised, Draped or Partially Debonded Strands

When straight strands are bonded for the full length of a prestressed girder, the tensile and compressive stresses near the ends of the girder will likely exceed the allowable service limit state stresses. This occurs because the strand pattern is designed for stresses at or near midspan, where the dead load moment is highest and best able to balance the effects of the prestress. Near the ends of the girder this dead load moment approaches zero and is less able to balance the prestress force. This results in tensile stresses in the top of the girder and compressive stresses in the bottom of the girder. The allowable initial tensile and compressive stresses are presented in the first two bullet points of 19.3.3.11. These stresses are a function of f'_{ci}, the compressive strength of concrete at the time of prestress force transfer. Transfer and development lengths should be considered when checking stresses near the ends of the girder.

The designer should start with a straight (raised), fully bonded strand pattern. If this overstresses the girder near the ends, the following methods shall be utilized to bring the girder within the allowable stresses. These methods are listed in order of preference and discussed in the following sections:

1. Use raised strand pattern (If excessive top flange reinforcement or if four or more additional strands versus a draped strand pattern are required, consider the draped strand alternative)

2. Use draped strand pattern

3. Use partially debonded strand pattern (to be used sparingly)

Only show one strand pattern per span (i.e. Do not show both raised and draped span alternatives for a given span).

A different girder spacing may need to be selected. It is often more economical to add an extra girder line than to maximize the number of strands and use debonding.

19.3.3.12.1 Raised Strand Patterns

Some of the standard strand patterns listed in the Standard Details show a raised strand pattern. Generally strands are placed so that the center of gravity of the strand pattern is as close as possible to the bottom of the girder. With a raised strand pattern, the center of gravity of the strand pattern is raised slightly and is a constant distance from the bottom of the girder for its entire length. Present practice is to show a standard raised arrangement as a preferred alternate to draping for short spans. For longer spans, debonding at the ends of the strands is an alternate (see 19.3.3.12.3). Use 0.6” strands for all raised patterns.

19.3.3.12.2 Draped Strand Patterns

Draping some of the strands is another available method to decrease stresses from prestress at the ends of the I-beam where the stress due to applied loads are minimum.

The typical strand profile for this technique is shown in Figure 19.3-1.

Bond breakers should only be applied to interior strands as girder cracking has occurred when they were applied to exterior strands. In computing bond breaker lengths, consideration is given to the theoretical stresses at the ends of the girder. These stresses are due entirely to prestress. As a result, the designer may compute a stress reduction based on certain strands having bond breakers. This reduction can be applied along the length of the debonded strands.

Partially debonded strands must adhere to the requirements listed in **LRFD [5.11.4.3]**. The list of requirements is as follows:

- The development length of partially debonded strands shall be calculated in accordance with **LRFD [5.11.4.2]** with $\kappa = 2.0$.
- The number of debonded strands shall not exceed 25% of the total number of strands.
- The number of debonded strands in any horizontal row shall not exceed 40% of the strands in that row.
- The length of debonding shall be such that all limit states are satisfied with consideration of the total developed resistance (transfer and development length) at any section being investigated.
- Not more than 40% of the debonded strands, or four strands, whichever is greater, shall have debonding terminated at any section.
- The strand pattern shall be symmetrical about the vertical axis of the girder. The consideration of symmetry shall include not only the strands being debonded but their debonded length as well, with the goal of keeping the center of gravity of the prestress force at the vertical centerline of the girder at any section. If the center of gravity of the prestress force deviates from the vertical centerline of the girder, the girder will twist, which is undesirable.
- Exterior strands in each horizontal row shall be fully bonded for crack control purposes.

19.3.3.13 Strength Limit State

The design factored positive moment is determined using the following equation:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

The Strength I limit state is applied to both simple and continuous span structures. See 17.2.4 for further information regarding loads and load combinations.

19.3.3.13.1 Factored Flexural Resistance

The nominal flexural resistance assuming rectangular behavior is given by **LRFD [5.7.3.2.3]** and **LRFD [5.7.3.2.2]**.

The section will act as a rectangular section as long as the depth of the equivalent stress block, a, is less than or equal to the depth of the compression flange (the structural deck thickness). Per **LRFD [5.7.3.2.2]**:

$$a = c\beta_1$$

Where:

c	=	Distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded (in)
β_1	=	Stress block factor **LRFD [5.7.2.2]**

By neglecting the area of mild compression and tension reinforcement, the equation presented in **LRFD [5.7.3.1.1]** for rectangular section behavior reduces to:

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$

Where:

A_{ps}	=	Area of prestressing steel (in^2)
f_{pu}	=	Specified tensile strength of prestressing steel (ksi)
f'_c	=	Compressive strength of the flange ($f'_{c(deck)}$ for rectangular section) (ksi)
b	=	Width of compression flange (in)
k	=	0.28 for low relaxation strand per **LRFD [C5.7.3.1.1]**
d_p	=	Distance from extreme compression fiber to the centroid of the prestressing tendons (in)
α_1	=	Stress block factor; equals 0.85 (for $f'_C \leq 10.0$ ksi) **LRFD [5.7.2.2]**

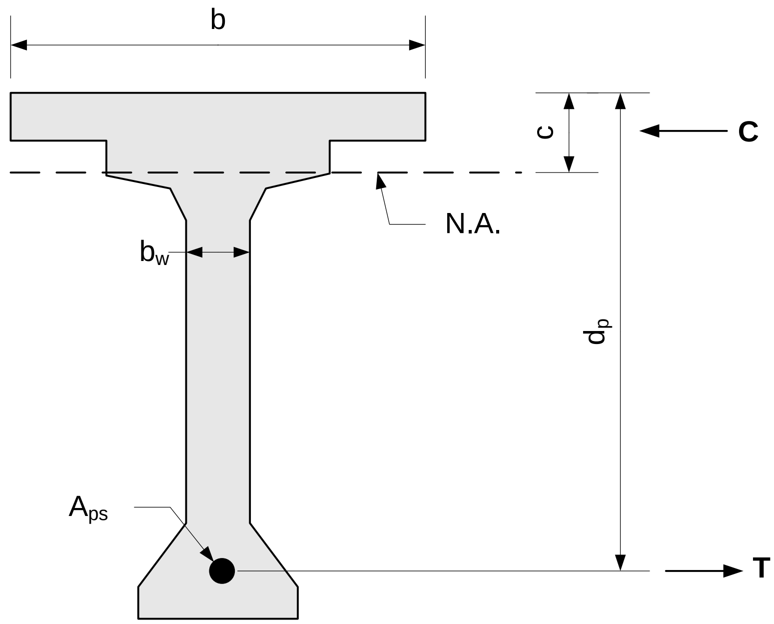

Figure 19.3-3
Depth to Neutral Axis, c

Verify that rectangular section behavior is allowed by checking that the depth of the equivalent stress block, a, is less than or equal to the structural deck thickness. If it is not, then T-section behavior provisions should be followed. If the T-section provisions are used, the compression block will be composed of two different materials with different compressive strengths. In this situation, **LRFD [C5.7.2.2]** recommends using β_1 and α_1 corresponding to the lower f'_c. The following equation for c shall be used for T-section behavior: **LRFD [5.7.3.1.1]**

$$c = \frac{A_{ps}f_{pu} - \alpha_1 f'_c(b - b_w)h_f}{\alpha_1 f'_c \beta_1 b_w + kA_{ps}\dfrac{f_{pu}}{d_p}}$$

Where:

b_w = Width of web (in) – use the top flange width if the compression block does not extend below the haunch.

h_f = Depth of compression flange (in)

The factored flexural resistance presented in **LRFD [5.7.3.2.2]** is simplified by neglecting the area of mild compression and tension reinforcement. Furthermore, if rectangular section behavior is allowed, then b_w = b, where b_w is the web width as shown in Figure 19.3-3. The equation then reduces to:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

Where:

M_r	=	Factored flexural resistance (kip-in)
ϕ	=	Resistance factor
f_{ps}	=	Average stress in prestressing steel at nominal bending resistance (refer to **LRFD [5.7.3.1.1]**) (ksi)

If the T-section provisions must be used, the factored moment resistance equation is then:

$$M_r = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + \alpha_1 \phi f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Where:

h_f	=	Depth of compression flange with width, b (in)

The engineer must then verify that M_r is greater than or equal to M_u.

<u>WisDOT exception to AASHTO:</u>

WisDOT standard prestressed concrete girders and strand patterns are tension-controlled. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required when the standard girders and strand patterns are used, and $\phi = 1$.

19.3.3.13.2 Minimum Reinforcement

Per **LRFD [5.7.3.3.2]**, the minimum amount of prestressed reinforcement provided shall be adequate to develop an M_r at least equal to the lesser of M_{cr}, or $1.33M_u$.

M_{cr} is the cracking moment, and is given by:

$$M_{cr} = \gamma_3 \left[S_c \left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) - 12M_{dnc} \left[(S_c/S_{nc}) - 1 \right] \right]$$

Where:

S_c	=	Section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in^3)
f_r	=	Modulus of rupture (ksi)
f_{cpe}	=	Compressive stress in concrete due to effective prestress forces only (after losses) at extreme fiber of section where tensile stress

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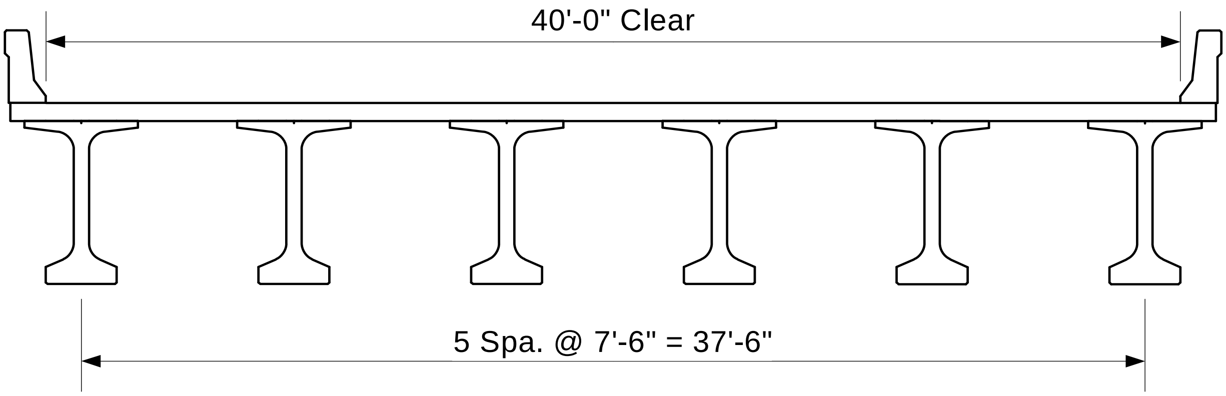
E19-1 Single Span Bridge, 72W" Prestressed Girders - LRFD

This example shows design calculations for a single span prestressed gider bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2015 Interim)***

E19-1.1 Design Criteria



$L := 146$	center to center of bearing, ft
$L_g := 147$	total length of the girder (the girder extends 6 inches past the center of bearing at each abutment).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{ci} := 6.8$	girder initial concrete strength, ksi New limit for release strength.
$f'_{cd} := 4$	deck concrete strength, ksi
$f_{pu} := 270$	low relaxation strand, ksi
$d_b := 0.6$	strand diameter, inches
$A_s := 0.217$	area of strand, in^2
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$skew := 20$	skew angle, degrees
$E_s := 28500$	ksi, Modulus of Elasticity of the Prestressing Strands
$w_c := 0.150$	kcf

E19-1.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}}$$ $$\boxed{E_{beam8} = 6351}$$ $$E_B := E_{beam8}$$

$$E_D := E_{deck4}$$

$$n := \frac{E_B}{E_D}$$ $$\boxed{n = 1.540}$$

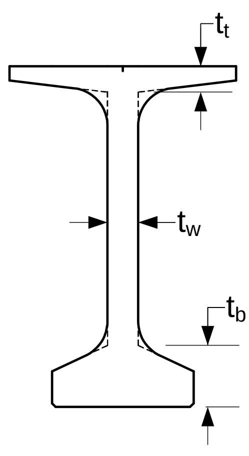
Note that this value of E_B is used for strength, composite section property, and long term deflection (deck and live load) calculations.

The value of the modulus of elasticity at the time of release is calculated in accordance with **LRFD [C5.4.2.4]**. This value of E_{ct} is used for loss and instantaneous deflection (due to prestress and dead load of the girder) calculations.

$$E_{beam6.8} := 33000 \cdot w_c^{1.5} \cdot \sqrt{f'_{ci}}$$ $$\boxed{E_{beam6.8} = 4999}$$ $$E_{ct} := E_{beam6.8}$$

E19-1.3 Section Properties

72W Girder Properties:



$w_{tf} := 48$	in		
$t_t := 5.5$	in		
$t_w := 6.5$	in		
$t_b := 13$	in		
$ht := 72$	in		
$b_w := 30$	width of bottom flange, in		
$A_g := 915$	in^2		
$r_{sq} := 717.5$	in^2	$y_b := -34.87$	in
$I_g := 656426$	in^4	$S_t := 17680$	in^3
$y_t := 37.13$	in	$S_b := -18825$	in^3

$e_g := y_t + 2 + \frac{t_{se}}{2}$ $\boxed{e_g = 42.88}$ in

Web Depth: $d_w := ht - t_t - t_b$ $\boxed{d_w = 53.50}$ in

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ **LRFD [Eq 4.6.2.2.1-1]** $\boxed{K_g = 3600866}$ in^4

E19-1.4 Girder Layout

Chapter 19 suggests that at a 146 foot span, the girder spacing should be 8'-6" with 72W girders.

$S := 8.5$ ft

Assume a minimum overhang of 2.5 feet (2 ft flange + 6" overhang), $s_{oh} := 2.5$

$n_{spa} := \frac{w_b - 2 \cdot s_{oh}}{S}$ $\boxed{n_{spa} = 4.412}$

Use the next lowest integer: $n_{spa} := \text{ceil}\left(n_{spa}\right)$ $\boxed{n_{spa} = 5}$

Number of girders: $ng := n_{spa} + 1$ $\boxed{ng = 6}$

Overhang Length: $s_{oh} := \frac{w_b - S \cdot n_{spa}}{2}$ $\boxed{s_{oh} = 0.00}$ ft

Recalculate the girder spacing based on a minimum overhang, $s_{oh} := 2.5$

$S := \frac{w_b - 2 \cdot s_{oh}}{n_{spa}}$ $\boxed{S = 7.50}$ ft

E19-1.5 Loads

$w_g := 0.953$ weight of 72W girders, klf

$w_d := 0.100$ weight of 8-inch deck slab (interior), ksf

$w_h := 0.125$ weight of 2.5-in haunch, klf

$w_{di} := 0.460$ weight of diaphragms on interior girder (assume 2), kips

$w_{dx} := 0.230$ weight of diaphragms on exterior girder, kips

$w_{ws} := 0.020$ future wearing surface, ksf

$w_p = 0.387$ weight of parapet, klf

$$\frac{f_{bp}}{1 - loss_{\%}} = \frac{T_o}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

$$f_{bpi\_1} := \frac{f_{bp}}{1 - \frac{loss_{\%}}{100}}$$ $\boxed{f_{bpi\_1} = 6.149}$ ksi

desired bottom initial prestress (before losses)

If we use the actual allowable tensile stress in the concrete, the desired bottom initial prestress is calculated as follows:

The allowable tension, from **LRFD [5.9.4.2]**, is:

$f_{tall} := 0.19 \cdot \sqrt{f'_c} \le 0.6$ ksi $\boxed{f_{tall} = 0.537}$ ksi

The desired bottom initial prestress (before losses):

$f_{bpi\_2} := f_{bpi\_1} - f_{tall}$ $\boxed{f_{bpi\_2} = 5.612}$ ksi

Determine the stress effects for different strand patterns on the 72W girder:

$A_s = 0.217$ in^2

$f'_s := 270000$ psi

$f_s := 0.75 \cdot f'_s$ $\boxed{f_s = 202500}$ psi

$$P := A_s \cdot \frac{f_s}{1000}$$ $\boxed{P = 43.943}$ kips

$$f_{bpi} := \frac{P \cdot N}{A_g} \cdot \left(1 + e \cdot \frac{y_b}{r_{sq}}\right)$$ (bottom initial prestress - before losses)

The values of f_{bpi} for various strand patterns is shown in the following table.

72W Stress Effects		
Pi (per strand) = 43.94 kips		
No. Strands	e (in)	bottom stress (ksi)
36	-31.09	4.3411
38	-30.98	4.5726
40	-30.87	4.8030
42	-30.77	5.0333
44	-30.69	5.2648
46	-30.52	5.4858
48	-30.37	5.7075
50	-30.23	5.9290
52	-30.10	6.1504

Solution:
Try $ns := 46$ strands, 0.6 inch diameter.
Initial prestress at bottom $f_{bpi} := 5.4858$ ksi,
Eccentricity, $e_s := -30.52$ inches; actual tension should be less than allowed.

E19-1.10.2 Prestress Loss Calculations

The loss in prestressing force is comprised of the following components:

1) Elastic Shortening (ES), shortening of the beam as soon as prestress is applied.

2) Shrinkage (SH), shortening of the concrete as it hardens, time function.

3) Creep (CR), slow shortening of concrete due to permanent compression stresses in the beam, time function.

4) Relaxation (RE), the tendon slowly accommodates itself to the stretch and the internal stress drops with time

E19-1.10.2.1 Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$$T_{oi} := ns \cdot f_{tr} \cdot A_s \qquad \boxed{= 46 \cdot 0.75 \cdot 270 \cdot 0.217 = 2021} \quad \text{kips}$$

The ES loss estimated above was: $\Delta f_{pES\_est} = 18.0$ ksi, or $ES_{loss} = 8.889$ %. The resulting force in the strands after ES loss:

$$T_o := \left(1 - \frac{ES_{loss}}{100}\right) \cdot T_{oi} \qquad \boxed{T_o = 1842} \quad \text{kips}$$

If we assume all strands are straight we can calculate the initial elastic shortening loss;

$$f_{cgp} := \frac{T_o}{A_g} + (T_o \cdot e_s) \cdot \frac{e_s}{I_g} + M_g \cdot 12 \cdot \frac{e_s}{I_g} \qquad \boxed{f_{cgp} = 3.190} \quad \text{ksi}$$

$$\boxed{E_{ct} = 4999} \quad \text{ksi}$$

$$E_p := E_s \qquad \boxed{E_p = 28500} \quad \text{ksi}$$

$$\Delta f_{pES} := \frac{E_p}{E_{ct}} \cdot f_{cgp} \qquad \boxed{\Delta f_{pES} = 18.185} \quad \text{ksi}$$

Check at the girder and deck at midspan:

$\boxed{e_s = -30.52}$ inches

Initial condition at transfer: $\boxed{f_{tiall} = -0.200}$ ksi $\boxed{f_{ciall} = 4.080}$ ksi

Top of girder stress (Service 3):

$$f_{ti} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_t} + \frac{M_g \cdot 12}{S_t}$$ $\boxed{f_{ti} = 0.582}$ ksi

Is f_{ti} greater than f_{tiall}? $\boxed{check = \text{"OK"}}$

Bottom of girder stress (Service 1):

$$f_{bi} := \frac{T_o}{A_g} + \frac{T_o \cdot e_s}{S_b} + \frac{M_g \cdot 12}{S_b}$$ $\boxed{f_{bi} = 3.353}$ ksi

Is f_{bi} less than f_{ciall}? $\boxed{check = \text{"OK"}}$

Final condition:

Allowable Stresses,**LRFD [5.9.4.2]**:

There are two compressive stress limits: (Service 1) **LRFD [5.9.4.2.1]**

$f_{call1} := 0.45 \cdot f'_c$ PS + DL $\boxed{f_{call1} = 3.600}$ ksi

$f_{call2} := 0.60 \cdot f'_c$ LL + PS + DL $\boxed{f_{call2} = 4.800}$ ksi

(Service 3) **LRFD [5.9.4.2.2]** (Moderate Corrosion Condition)

tension: $f_{tall} := -0.19 \cdot \sqrt{f'_c}$ $|f_{tall}| \leq 0.6$ ksi $\boxed{f_{tall} = -0.537}$ ksi

Allowable Stresses (Fatigue),**LRFD [5.5.3]**:

Fatige compressive stress limit:

$f_{call\_fat} := 0.40 \cdot f'_c$ LLfat + 1/2(PS + DL) $\boxed{f_{call\_fat} = 3.200}$ ksi

Top of girder stress (Service 1):

$$f_{t1} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{\left(M_{DLc} + M_{DWc}\right) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t1} = 2.465}$ ksi

$$f_{t2} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t} + \frac{\left(M_{DLc} + M_{DWc} + M_{LL}\right) \cdot 12}{S_{cgt}}$$

$\boxed{f_{t2} = 3.177}$ ksi

Is f_t less than f_{call}?

$\boxed{check1 = \text{"OK"}}$

$\boxed{check2 = \text{"OK"}}$

Top of girder stress (Fatigue 1):

$$f_{tfat} := \frac{1}{2}\left(\frac{T}{A_g} + \frac{T \cdot e_s}{S_t} + \frac{M_{DLnc} \cdot 12}{S_t}\right) + \frac{\left[\frac{1}{2}\left(M_{DLc} + M_{DWc}\right) + M_{LLfat}\right] \cdot 12}{S_{cgt}}$$

$\boxed{f_{tfat} = 1.434}$ ksi

Is f_{tfat} less than f_{call_fat}?

$\boxed{check = \text{"OK"}}$

Bottom of girder stress (Service 3):

$$f_b := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b} + \frac{M_{nc} \cdot 12}{S_b} + \frac{\left(M_{s3} - M_{nc}\right) \cdot 12}{S_{cgb}}$$

$\boxed{f_b = -0.302}$ ksi

Is f_{tb} greater than f_{tall}?

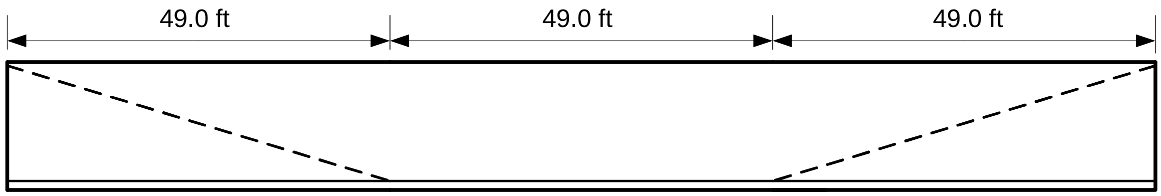
$\boxed{check = \text{"OK"}}$

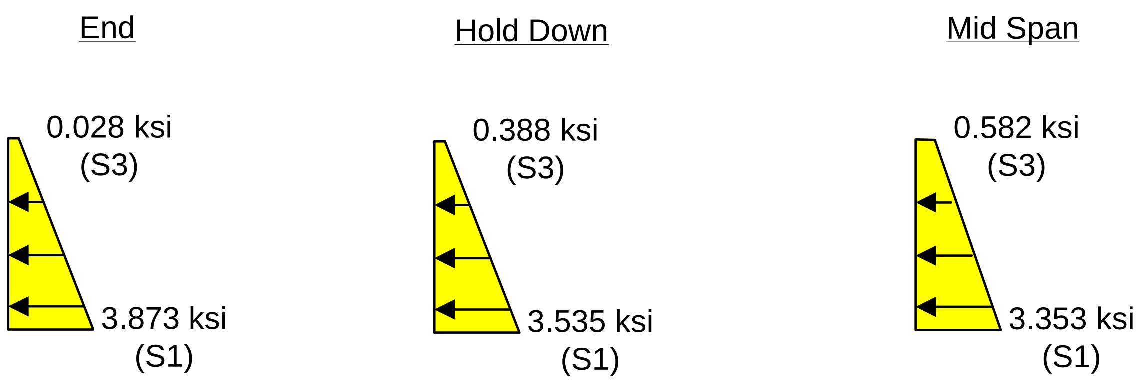
Top of deck stress (Service 1):

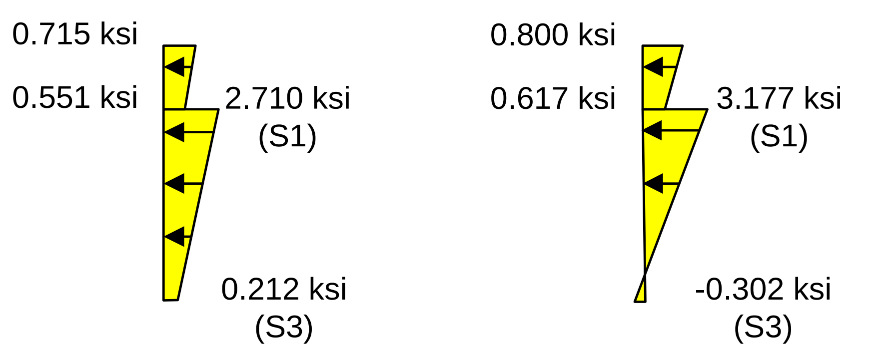
$$f_{dall} := 0.40 \cdot f'_{cd}$$

$\boxed{f_{dall} = 1.600}$ ksi

Summary of Design Stresses:







Initial Allowable:

$compression := f_{ciall} = 4.08$ ksi

Final Allowable:

$compression_1 := f_{call1} = 3.6$ ksi

$compression_2 := f_{call2} = 4.8$ ksi

$compression\_fatigue := f_{call\_fat} = 3.2$ ksi

$tension = f_{tall} = -0.537$ ksi **All stresses are acceptable!**

E19-1.11 Calculate Jacking Stress

The fabricator is responsible for calculation of the jacking force. See **LRFD [5.9.3]** for equations for low relaxation strands.

E19-1.12 Flexural Strength Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.7.3.1.1]**:

$\boxed{f_{pe} = 160}$ ksi $\boxed{0.5 \cdot f_{pu} = 135}$ ksi

Is 0.5*f_{pu} less than f_{pe}? $\boxed{check = \text{"OK"}}$

Then at failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

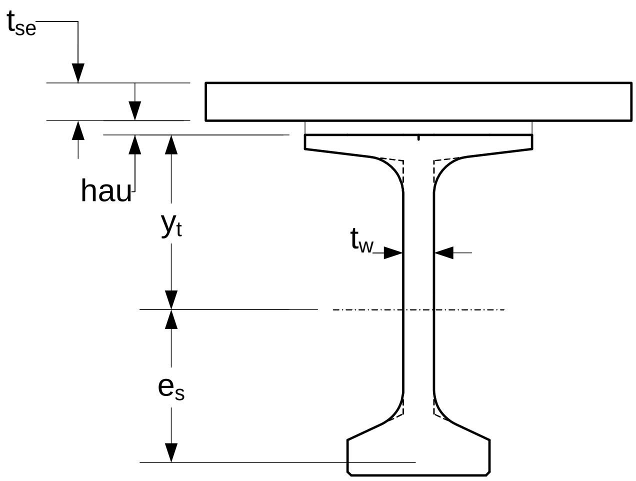
where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD Table [C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assumed dimensions:



Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := ns \cdot A_s$ $\boxed{A_{ps} = 9.98}$ in^2

$b := w_e$ $\boxed{b = 90.00}$ in

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \le 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_{cd} - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.850}$

$d_p := y_t + hau + t_{se} - e_s$ $\boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 9.99}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 8.49}$ in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$ depth of compression flange $\boxed{h_f = 7.500}$ in

$w_{tf} = 48.00$ width of top flange, inches

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.937}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 9.30}$ in

This is within the depth of the haunch (9.5 inches). Therfore our assumption is OK.

Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 259.283}$ ksi

$T_u := f_{ps} \cdot A_{ps}$ $\boxed{T_u = 2588}$ kips

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.7.3.2]; [5.7.3.2.2]**

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot \left(b - w_{tf}\right) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 15717}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 15717}$ kip-ft

The required capacity:

Interior Girder Moment $\boxed{M_{str} = 12449}$ kip-ft

Exterior Girder Moment $\boxed{M_{strx} = 11183}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]** for the interior girder:

$\boxed{1.33 \cdot M_{str} = 16558}$ kip-ft

$f_r := 0.24 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\boxed{f_r = 0.679}$ ksi

$$f_{cpe} := \frac{T}{A_g} + \frac{T \cdot e_s}{S_b}$$ $\boxed{f_{cpe} = 4.348}$ ksi

$M_{dnc} := M_{nc}$ $\boxed{M_{dnc} = 4887}$ kip-ft

$S_c := -S_{cgb}$ $\boxed{S_c = 24681}$ in^3

$S_{nc} := -S_b$ $\boxed{S_{nc} = 18825}$ in^3

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete structures

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$ $\boxed{M_{cr} = 10551}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and $1.33{*}M_{str}$? $\boxed{check = \text{"OK"}}$

The moment capacity looks good, with some over strength for the interior girder. However, we must check the capacity of the exterior girder since the available flange width is less.

Check the exterior girder capacity:

The effective flange width for exterior girder is calculated in accordance with **LRFD [4.6.2.6]** as one half the effective width of the adjacent interior girder plus the overhang width :

$$w_{ex\_oh} := s_{oh} \cdot 12 \qquad \boxed{w_{ex\_oh} = 30.0} \text{ in}$$

$$w_{ex} := \frac{w_e}{2} + w_{ex\_oh} \qquad \boxed{w_{ex} = 75.00} \text{ in}$$

$b_x := w_{ex}$ effective deck width of the compression flange.

Calculate the neutral axis location for a flanged section:

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c_x := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot \left(b_x - w_{tf}\right) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot w_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c_x = 13.51} \text{ in}$$

$$a_x := \beta_1 \cdot c_x \qquad \boxed{a_x = 11.49} \text{ in}$$

Now calculate the effective tendon stress at ultimate:

$$f_{ps\_x} := f_{pu} \cdot \left(1 - k \cdot \frac{c_x}{d_p}\right) \qquad \boxed{f_{ps\_x} = 256.759} \text{ ksi}$$

The nominal moment capacity of the composite section (exterior girder) ignoring the increased strength of the concrete in the girder flange:

$$M_{n\_x} := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a_x}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot \left(b_x - w_{tf}\right) \cdot h_f \cdot \left(\frac{a_x}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$$\boxed{M_{n\_x} = 15515} \text{ kip-ft}$$

$$M_{r\_x} := \phi_f \cdot M_{n\_x} \qquad \boxed{M_{r\_x} = 15515} \text{ kip-ft}$$

$\boxed{1.33M_{strx} = 14874}$ kip-ft

Is M_{r_x} greater than 1.33*M_{strx}? $\boxed{check = "OK"}$

Since M_{r_x} is greater than 1.33*M_{strx}, the check for M_{cr} does not need to be completed.

E19-1.13 Shear Analysis

A separate analysis must be conducted to estimate the total shear force in each girder for shear design purposes.

Calculate the shear distribution to the girders, **LRFD [Table 4.6.2.2.3a-1]**:

Interior Beams:

One lane loaded:

$$g_{vi1} := 0.36 + \frac{S}{25} \qquad \boxed{g_{vi1} = 0.660}$$

Two or more lanes loaded:

$$g_{vi2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \boxed{g_{vi2} = 0.779}$$

$$g_{vi} := \max\left(g_{vi1}, g_{vi2}\right) \qquad \boxed{g_{vi} = 0.779}$$

Note:The distribution factors above include the multiple lane factor. *The skew correction factor, as now required by a WisDOT policy item for all girders, is omitted. This example is not yet revised.*

Exterior Beams:

Two or more lanes loaded:

The distance from the centerline of the exterior beam to the inside edge of the parapet, $d_e = 1.25$ feet.

$$e_v := 0.6 + \frac{d_e}{10} \qquad \boxed{e_v = 0.725}$$

$$g_{vx2} := e_v \cdot g_{vi} \qquad \boxed{g_{vx2} = 0.565}$$

With a single lane loaded, we use the lever rule (same as before). Note that the multiple presence factor has already been applied to g_{x2}..

$$g_{vx1} := g_{x1} \quad = e \cdot g_i \qquad \boxed{g_{vx1} = 0.600}$$

$\boxed{d_b = 0.600}$ in

$$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b \qquad \boxed{l_d = 146.2} \text{ in}$$

The transfer length may be taken as: $l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.250$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} + \frac{L_{crit} \cdot 12 - l_{tr}}{l_d - l_{tr}} \cdot \left(f_{ps} - f_{pe}\right) \qquad \boxed{f_{pu\_crit} = 195} \text{ ksi}$$

For rectangular section behavior:

LRFD [5.7.2.2] $\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.850}$

$$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \frac{f_{pu\_crit}}{d_{p\_crit}}} \qquad \boxed{c = 7.276} \text{ in}$$

$$a_{crit} := \beta_1 \cdot c \qquad \boxed{a_{crit} = 6.184} \text{ in}$$

Calculation of shear depth based on refined calculations of e_s and a:

$$d_{v\_crit} := -e_{s\_crit} + y_t + hau + t_{se} - \frac{a_{crit}}{2} \qquad \boxed{d_{v\_crit} = 64.65} \text{ in}$$

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**). Note, the value of V_p does not equal zero in the calculation of V_{cw}.

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kips)

V_i = factored shear force at section due to externally applied loads (Live Loads) occurring simultaneously with M_{max} (kips). (Not necessarily equal to V_u.)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in)

M_{max} = maximum factored moment at section due to externally applied loads (Live Loads) (kip-in)

M_{dnc} = total unfactored dead load moment acting on the noncomposite section (kip-ft)

Values for the following moments and shears are at the critical section, $L_{crit} = 6.25$ feet from the end of the girder at the abutment.

$V_d = 141$ kips

$V_i = 136$ kips

$M_{dnc} = 740$ kip-ft

$M_{max} = 837$ kip-ft

However, the equations below require the value of M_{max} to be in kip-in:

$M_{max} = 10048$ kip-in

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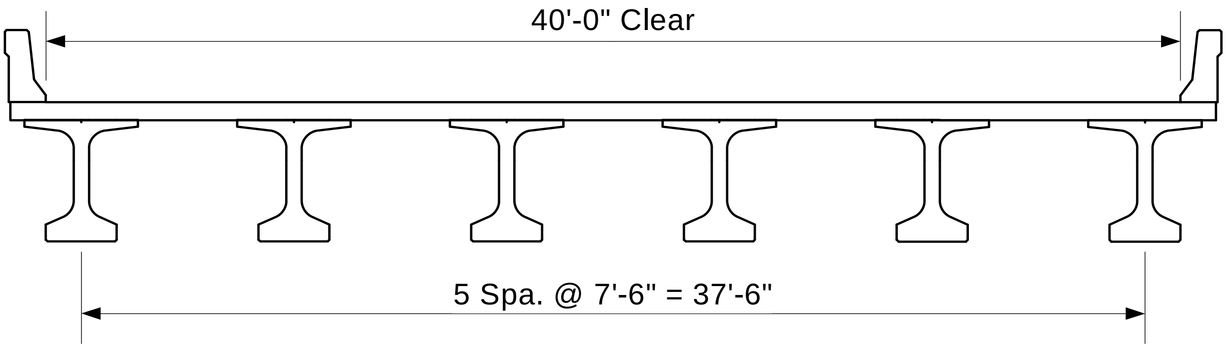
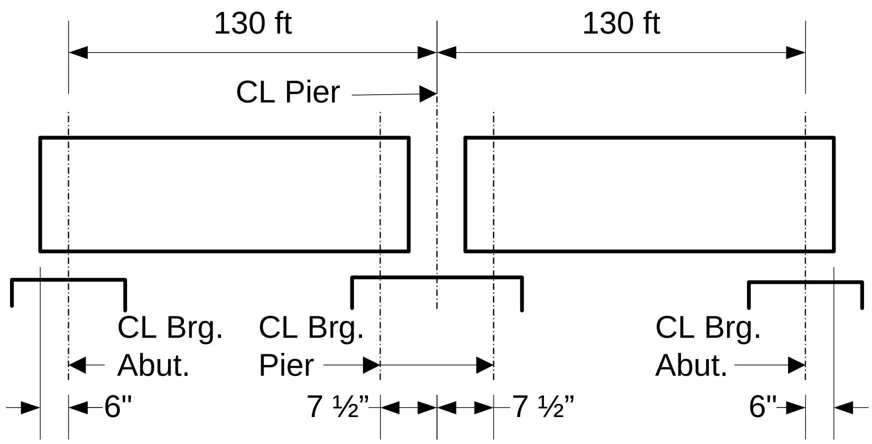
E19-2 Two-Span 54W" Girder, Continuity Reinforcement - LRFD

This example shows design calculations for the continuity reinforcement for a two span prestressed girder bridge. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter.***(Example is current through LRFD Seventh Ed. - 2015 Int.)***

E19-2.1 Design Criteria





$L := 130$	center of bearing at abutment to CL pier for each span, ft
$L_g := 130.375$	total length of the girder (the girder extends 6 inches past the center of bearing at the abutment and 1.5" short of the center line of the pier).
$w_b := 42.5$	out to out width of deck, ft
$w := 40$	clear width of deck, 2 lane road, 3 design lanes, ft
$f'_c := 8$	girder concrete strength, ksi
$f'_{cd} := 4$	deck concrete strength, ksi
$f_y := 60$	yield strength of mild reinforcement, ksi

interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L}$$ $\boxed{w_{dlii} = 1.687}$ klf

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng}$$ $\boxed{w_p = 0.129}$ klf

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng}$$ $\boxed{w_{ws} = 0.133}$ klf

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E19-2.5.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = truck + lane **LRFD [3.6.1.3.1]**
truck pair + lane

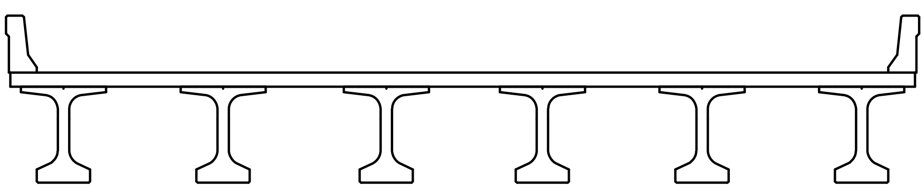
DLA of 33% applied to truck or tandem, but not to lane per **LRFD [3.6.2.1].**

For Fatigue 1:

HL-93 truck (no lane) with 15% DLA and 30 ft rear axle spacing per **LRFD [3.6.1.4.1].**

E19-2.6 Load Distribution to Girders

In accordance with **LRFD [Table 4.6.2.2.1-1]**,
this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1}$$

Criteria for using distribution factors - Range of Applicability per **LRFD [Table 4.6.2.2.2b-1]**.

$$\text{DeckSpan} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 3.5 \le S \le 16 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{DeckThickness} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 4.5 \le t_{se} \le 12 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{BridgeSpan} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 20 \le L \le 240 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{NoBeams} := \left| \begin{array}{ll} \text{"OK"} & \text{if } ng \ge 4 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$\text{LongitStiffness} := \left| \begin{array}{ll} \text{"OK"} & \text{if } 10000 \le K_g \le 7000000 \\ \text{"NG"} & \text{otherwise} \end{array} \right.$$

$$x := \begin{pmatrix} S & \text{DeckSpan} \\ t_{se} & \text{DeckThickness} \\ L & \text{BridgeSpan} \\ ng & \text{NoBeams} \\ K_g & \text{LongitStiffness} \end{pmatrix} \qquad x = \begin{pmatrix} 7.5 & \text{"OK"} \\ 7.5 & \text{"OK"} \\ 130.0 & \text{"OK"} \\ 6.0 & \text{"OK"} \\ 1868972.4 & \text{"OK"} \end{pmatrix}$$

E19-2.6.1 Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}{}^3}\right)^{0.1} \qquad \boxed{g_{i1} = 0.427}$$

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

$$R_u := \frac{M_u \cdot 12}{\phi_f \cdot b_w \cdot d_e^2} \qquad \boxed{R_u = 0.532} \text{ ksi}$$

$$\rho := 0.85 \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'_c}}\right) \qquad \boxed{\rho = 0.00925}$$

$$A_s := \rho \cdot b_w \cdot d_e \qquad \boxed{A_s = 16.74} \text{ in}^2$$

This reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches. The required continuity reinforcement in in^2/ft is equal to:

$$As_{req} := \frac{A_s}{\frac{w_e}{12}} \qquad \boxed{As_{req} = 2.232} \text{ in}^2\text{/ft}$$

From Chapter 17, Table 17.5-3, for a girder spacing of $S = 7.5$ feet and a deck thickness of $t_s = 8.0$ inches, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement shall be placed at 1/2 of this bar spacing, .

#9 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 2.82}$ in^2/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12} \qquad \boxed{As = 21.18} \text{ in}^2$$

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

Assume $f_s = f_y$ **LRFD [5.7.2.2]** $\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi)

$$a := \frac{As \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c} \qquad \boxed{a = 6.228} \text{ in}$$

This is within the thickness of the bottom flange height of 7.5 inches.

If $\frac{c}{d_s} \leq 0.6$ for ($f_y = 60$ ksi) **LRFD [5.7.2.1]**, the reinforcement has yielded and the assumption is correct.

LRFD [5.7.2.2] $\beta_1 := 0.65$; $c := \frac{a}{\beta_1}$ $\boxed{c = 9.582}$ in

$\frac{c}{d_s} = 0.16 < 0.6$ therefore, the reinforcement will yield

$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12}$ $\boxed{M_n = 6056}$ kip-ft

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 5451}$ kip-ft

$\boxed{M_u = 4358}$ kip-ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r := 0.24 \cdot \sqrt{f'_{cd}}$ $\boxed{f_r = 0.480}$ ksi

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 1709}$ kip-ft

$\boxed{1.33 \cdot M_u = 5796}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho := \frac{As}{b_w \cdot d_e}$ $\boxed{\rho = 0.01170}$

$n := \frac{E_s}{E_B}$ $\boxed{n = 4.566}$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$ $\boxed{k = 0.278}$

$j := 1 - \frac{k}{3}$ $\boxed{j = 0.907}$

$$\Delta f := n \cdot \frac{\left|M_{LLfatigue}\right| \cdot y_{rb}}{I_{cr}} \cdot 12$$ $\boxed{\Delta f = 3.488}$ ksi

$\boxed{\gamma f_{LL} \cdot \Delta f = 5.232}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{\text{check} = \text{"OK"}}$

E19-2.13 Bar Cut Offs

The first cut off is located where half of the continuity reinforcement satisfies the moment diagram. Non-composite moments from the girder and the deck are considered along with the composite moments when determining the Strength I moment envelope. (It should be noted that since the non-composite moments are opposite in sign from the composite moments in the negative moment region, the minimum load factor shall be applied to the non-composite moments.) Only the composite moments are considered when checking the Service and Fatigue requirements.

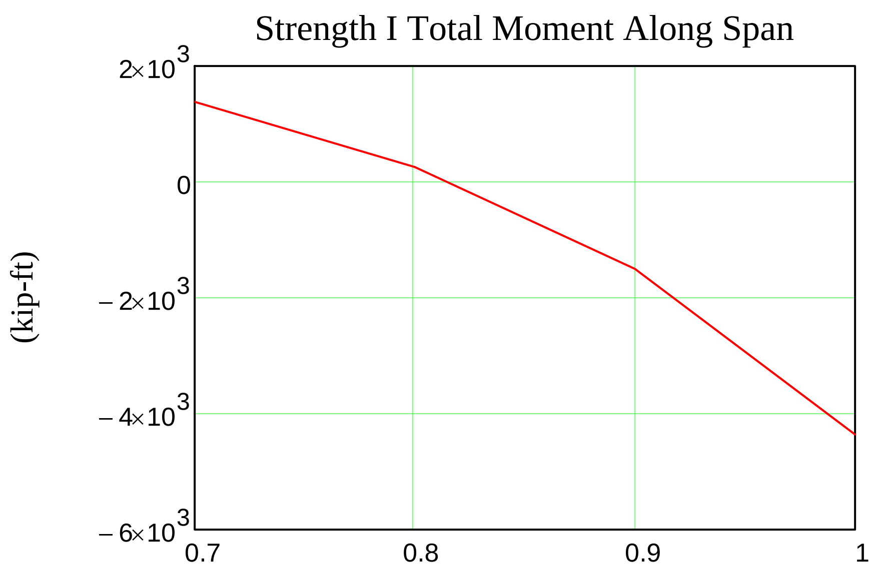
$$spa' := spa \cdot 2$$ $\boxed{spa' = 8.50}$ in

$$As' := \frac{As}{2}$$ $\boxed{As' = 10.588}$ in^2

$$a' := \frac{As' \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a' = 3.11}$ in

$$M_{n'} := As' \cdot f_y \cdot \left(d_e - \frac{a'}{2}\right) \cdot \frac{1}{12}$$ $\boxed{M_{n'} = 3111}$ kip-ft

$M_{r'} := \phi_f \cdot M_{n'}$ $\boxed{M_{r'} = 2799}$ kip-ft



Based on the moment diagram, try locating the first cut off at $cut_1 := 0.90$ span. Note that the Service I crack control requirements control the location of the cut off.

$\boxed{M_{r'} = 2799}$ kip-ft

$\boxed{Mu_{cut1} = 1501}$ kip-ft

$\boxed{Ms_{cut1} = 1565}$ kip-ft

Is Mu_{cut1} less than $M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 1709}$ kip-ft

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E19-3 Box Section Beam

This example shows design calculations for a single span prestressed box multi-beam bridge having a 2" concrete overlay and is designed for a 20 pound per square foot future wearing surface. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2015 Interim)***

E19-3.1 Preliminary Structure Data

Design Data
A-1 Abutments at both ends
Skew: 0 degrees
Live Load: HL-93
Roadway Width: 28 ft. minimum clear

$L := 44$	Span Length, single span, ft
$L_g := 44.5$	Girder Length, the girder extends 3" past the CL bearing at each abutment, single span, ft
$N_L := 2$	Number of design lanes
$t_{overlay} := 2$	Minimum overlay thickness, inches
$f_{pu} := 270$	Ultimate tensile strength for low relaxation strands, ksi
$d_s := 0.5$	Strand diameter, inches
$A_s := 0.1531$	Area of prestressing strands, in^2
$E_s := 28500$	Modulus of elasticity of the prestressing strands, ksi
$f'_c := 5$	Concrete strength (prestressed box girder), ksi
$f'_{ci} := 4.25$	Concrete strength at release, ksi
$K_1 := 1.0$	Aggregate correction factor
$w_c := 0.150$	Unit weight of concrete for box girder, overlay, and grout, kcf
$fy := 60$	Bar steel reinforcement, Grade 60, ksi.
$w_{rail} := 0.075$	Weight of Type "M" rail, klf
$Wh_{rail} := 0.42$	Width of horizontal members of Type "M" rail, feet
$\mu := 0.20$	Poisson's ratio for concrete, **LRFD [5.4.2.5]**

Based on past experience, the modulus of elasticity for the precast concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi for a concrete strength of 6 ksi. The values of E for different concrete strengths are calculated as follows:

$$E_{beam5} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}}$$ $\boxed{E_{beam5} = 5021}$ ksi

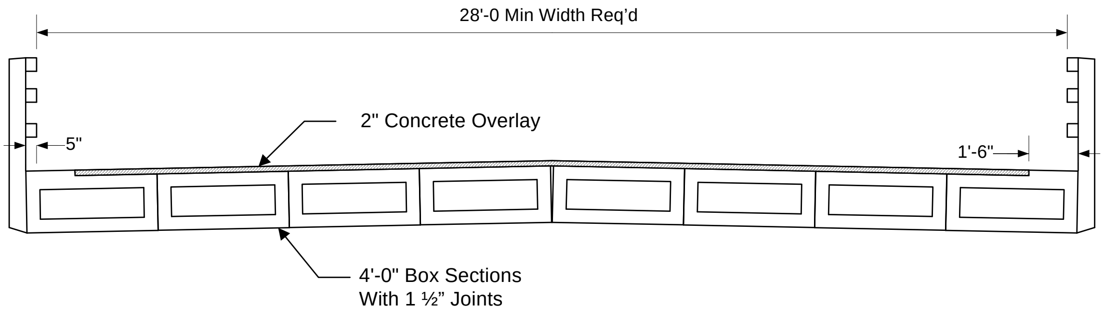
$E_B := E_{beam5}$

The modulus of elasticity at the time of release is calculated in accordance with **LRFD [C5.4.2.4]**.

$$E_{beam4.25} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_{ci}}$$ $\boxed{E_{beam4.25} = 3952}$ ksi

$E_{ct} := E_{beam4.25}$

Based on the preliminary data, Section 19.3.9 of this chapter and Table 19.3-3, select a 4'-0" wide pretensioned box section having a depth of 1'-9" (Section 3), as shown on Bridge Manual Standard 19.15. The actual total deck width provided is calculated below.



$n_{beams} := 8$

$n_{joints} := n_{beams} - 1$ $\boxed{n_{joints} = 7}$

$W_s := 4$ Width of section, ft

$W_j := 1.5$ Width of joints, inches

Overall width of the bridge, ft

$$W_b := n_{beams} \cdot W_s + n_{joints} \cdot \frac{W_j}{12}$$ $\boxed{W_b = 32.875}$ feet

Clear width of the bridge, ft

$W_{b\_clear} := W_b - 2 \cdot Wh_{rail}$ $\boxed{W_{b\_clear} = 32.035}$ feet

$W_{curb} := 1.5$ Width of curb on exterior girder (for steel rails), feet

$$S := W_s + \frac{W_j}{12}$$ Effective spacing of sections $\boxed{S = 4.125}$ feet

Section Properties, 4 ft x 1'-9" deep Box, Section 3

$D_s := 1.75$	Depth of section, ft
$A := 595$	Area of the box girder, in^2
$t_w := 5$	Thickness of each vertical element, in
$r_{sq} := 55.175$	in^2
$y_t := 10.5$	in
$y_b := -10.5$	in
$S_t := 3137$	Section modulus, in^3
$S_b := -3137$	Section modulus, in^3
$I := 32942$	Moment of inertia, in^4
$J := 68601$	St. Venant's torsional inertia, in^4

E19-3.2 Live Load Distribution

The live load distribution for adjacent box beams is calculated in accordance with **LRFD [4.6.2.2.2]**. Note that if the section does not fall within the applicability ranges, the lever rule shall be used to determine the distribution factor.

E19-3.2.1 Distribution for Moment

For interior beams, the live load moment distribution factor is calculated as indicated in **LRFD [Table 4.6.2.2.2b-1]** for cross section type "g" if connected only enough to prevent relative vertical displacement. This distribution factor applies regardless of the number of lanes loaded.

$$K := \sqrt{\frac{(1+\mu)\cdot I}{J}} \qquad \boxed{K = 0.759}$$

$$C := \min\left[K\cdot\left(\frac{W_b}{L}\right), K\right] \qquad \boxed{C = 0.567}$$

When C is less than 5:

$$D := 11.5 - N_L + 1.4\cdot N_L\cdot(1 - 0.2\cdot C)^2 \qquad \boxed{D = 11.701}$$

$$g_{int\_m} := \frac{S}{D} \qquad \boxed{g_{int\_m} = 0.353}$$

E19-3.9 Allowable Stress

Allowable stresses are determined for 2 sages for prestressed girders. Temporary allowable stresses are set for the loading stage at release of the prestressing strands. Final condition allowable stresses are checked at the end of 50 years of service.

E19-3.9.1 Temporary Allowable Stresses

The temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

$f_{ciall} := 0.60 \cdot f'_{ci}$ $\boxed{f_{ciall} = 2.550}$ ksi

In accordance with **LRFD [Table 5.9.4.1.2-1]**, the temporary allowable tension stress is calculated as follows (assume there is no bonded reinforcement):

$f_{tiall} := -\min\left(0.0948 \cdot \sqrt{f'_{ci}}, 0.2\right)$ $\boxed{f_{tiall} = -0.195}$ ksi

If bonded reinforcement is present in the top flange, the temporary allowable tension stress is calculated as follows:

$f_{tiall\_bond} := -0.24 \cdot \sqrt{f'_{ci}}$ $\boxed{f_{tiall\_bond} = -0.495}$ ksi

E19-3.9.2 Final Condition Allowable Stresses

Allowable Stresses,**LRFD [5.9.4.2]**:

There are two compressive service stress limits:

$f_{call1} := 0.45 \cdot f'_{c}$ PS + DL $\boxed{f_{call1} = 2.250}$ ksi

$f_{call2} := 0.60 \cdot f'_{c}$ LL + PS + DL $\boxed{f_{call2} = 3.000}$ ksi

There is one tension service stress limit:

$f_{tall} := -0.19 \cdot \sqrt{f'_{c}}$ LL + PS + DL $|f_{tall}| \le 0.6$ ksi $\boxed{f_{tall} = -0.425}$ ksi

There is one compressive fatigue stress limit:

$f_{call\_f} := 0.40 \cdot f'_{c}$ LLf + 1/2(PS + DL) $\boxed{f_{call\_f} = 2.000}$ ksi

E19-3.10 Preliminary Design Steps

The following steps are utilized to design the prestressing strands:

1) Design the amount of prestress to prevent tension at the bottom of the beam under the full load at center span after 50 years.

2) Calculate the prestress losses and check the girder stresses at mid span at the time of transfer.

3) Check resulting stresses at the critical sections of the girder at the time of transfer and after 50 years.

E19-3.10.1 Determine Amount of Prestress

Design the amount of prestress to prevent tension at the bottom of the beam under the full loac (at center span) after 50 years.

Near center span, after 50 years, T = the remaining effective prestress, aim for no tension at the bottom. Use Service I for compression and Service III for tension.

For this example, the exterior girder has the controlling moments.

Calculate the stress at the bottom of the beam due to the Service 3 loading:

$$f_b := \frac{M_{s3} \cdot 12}{S_b} \qquad \boxed{f_b = -1.867} \text{ ksi}$$

Stress at bottom due to prestressing:

$$f_{bp} = \frac{T}{A} \cdot \left(1 + e \cdot \frac{y_b}{r^2}\right)$$

and $f_{bp} := |f_b|$ desired final prestress.

We want this to balance out the tensile stress calculated above from the loading, i.e. an initial compression. The required stress due to prestress force at bottom of section to counteract the Service 3 loads:

$$\boxed{f_{bp} = 1.867} \text{ ksi}$$

E19-3.10.1.1 Estimate the Prestress Losses

At 50 years the prestress has decreased (due to CR, SH, RE):

The approximate method of estimated time dependent losses is used by WisDOT. The lump sum loss estimate, I-girder loss **LRFD [Table 5.9.5.3-1]**

Where PPR is the partial prestressing ratio, $PPR := 1.0$

$$f_{t2} := \frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{M_{s1} \cdot 12}{S_t}$$ LL + PS + DL $\boxed{f_{t2} = 1.719}$ ksi

$\boxed{check = "OK"}$

Bottom of girder stress (Compression - Service 1):

$$f_{b1} := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_b}$$ PS + DL $\boxed{f_{b1} = 0.958}$ ksi

$\boxed{check = "OK"}$

Bottom of girder stress (Tension - Service 3):

$$f_b := \frac{T}{A} + \frac{T \cdot e_s}{S_b} + \frac{M_{s3} \cdot 12}{S_b}$$ $\boxed{f_b = -0.051}$ ksi

$\boxed{check = "OK"}$

Top of girder stress (Compression - Fatigue 1):

$$f_{tf1} := \frac{1}{2} \cdot \left[\frac{T}{A} + \frac{T \cdot e_s}{S_t} + \frac{(M_{DC} + M_{DW}) \cdot 12}{S_t} \right] + \frac{M_{LLf} \cdot 12}{S_t}$$ 1/2(PS + DL) + LLf

$\boxed{f_{t1} = 0.459}$ ksi

$\boxed{check = "OK"}$

allowable stress (tension) $\boxed{f_{tall} = -0.425}$ ksi

allowable stress (compression) $\boxed{f_{call1} = 2.250}$ ksi

$\boxed{f_{call2} = 3.000}$ ksi

$\boxed{f_{call\_f} = 2.000}$ ksi

E19-3.11 Flexural Capacity at Midspan

Check f_{pe} in accordance with **LRFD [5.7.3.1.1]**:

$\boxed{f_{pe} = 172}$ ksi $\boxed{0.5 \cdot f_{pu} = 135}$ ksi

Is 0.5*f_{pu} less than f_{pe}? $\boxed{check = \text{"OK"}}$

Then at failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD [Table C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assume that the compression block is in the top section of the box. Calculate the capacity as if it is a rectangular section. The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := N \cdot A_s$ $\boxed{A_{ps} = 2.45}$ in^2

$b := W_s \cdot 12$ $\boxed{b = 48.00}$ in

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_c - 4\right) \cdot 0.05, 0.65\right]$ $\boxed{\beta_1 = 0.800}$

$d_p := y_t - e_s$ $\boxed{d_p = 18.75}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 3.82}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 3.06}$ in

Simplified Procedure for Prestressed and Nonprestressed Sections, **LRFD [5.8.3.4.3]**

$b_v := 2t_w$ $\boxed{b_v = 10.00}$ in

The critical section for shear is taken at a distance of d_v from the face of the support, **LRFD [5.8.3.2]**.

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of 0.9*d_e or 0.72h (inches). **LRFD [5.8.2.9]**

The first estimate of d_v is calculated as follows:

$d_v := -e_s + y_t - \frac{a}{2}$ $\boxed{d_v = 17.22}$ in

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$L_{crit} := \left(w_{brg} + d_v\right)\cdot\frac{1}{12}$ $\boxed{L_{crit} = 2.10}$ ft

The eccentricity of the strand group at the critical section is:

$\boxed{e_s = -8.25}$ in

Calculation of compression stress block:

$\boxed{d_p = 18.75}$ in

$\boxed{A_{ps} = 2.45}$ in^2

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.0$ for prestressed members with a depth less than 24 inches

$\boxed{d_s = 0.5}$ in

$l_d := K\cdot\left(f_{ps} - \frac{2}{3}\cdot f_{pe}\right)\cdot d_s$ $\boxed{l_d = 70.0}$ in

The transfer length may be taken as: $l_{tr} := 60\cdot d_s$ $\boxed{l_{tr} = 30.00}$ in

Since $L_{crit} = 2.102$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} \cdot \frac{L_{crit} \cdot 12}{l_{tr}}$$ $\boxed{f_{pu\_crit} = 145}$ ksi

$$T_{crit} := N \cdot A_s \cdot f_{pu\_crit}$$ $\boxed{T_{crit} = 354}$ kips

For rectangular section behavior:

$\boxed{\alpha_1 = 0.850}$ $\boxed{\beta_1 = 0.800}$

$$c_{crit} := \frac{A_{ps} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu\_crit}}{d_p}}$$ $\boxed{c_{crit} = 2.102}$ in

$$a_{crit} := \beta_1 \cdot c_{crit}$$ $\boxed{a_{crit} = 1.682}$ in

Calculation of shear depth based on refined calculations of a:

$$d_{v\_crit} := -e_s + y_t - \frac{a_{crit}}{2}$$ $\boxed{d_{v\_crit} = 17.91}$ in

This value matches the assumed value of d_v above. OK!

$$d_v := d_{v\_crit}$$

The location of the critical section from the end of the girder is:

$$L_{crit} := \left(w_{brg} + d_v\right) \cdot \frac{1}{12}$$ $\boxed{L_{crit} = 2.159}$ ft

The location of the critical section from the center line of bearing at the abutment is:

$$crit := L_{crit} - 0.25$$ $\boxed{crit = 1.909}$ ft

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

where $V_p := 0$ in the calculation of V_n, if the simplified procedure is used (**LRFD [5.8.3.4.3]**).

Note, the value of V_p does not equal zero in the calculation of V_{cw}.

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kips)

V_i = factored shear force at section due to externally applied loads (Live Loads) occurring simultaneously with M_{max} (kips). (Not necessarily equal to V_u.)

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E19-4 Lifting Check for Prestressed Girders, LRFD

This example shows design calculations for the lifting check for the girder in design example E19-1. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2015 Interim)***

E19-4.1 Design Criteria

$L_{girder} := 146$	feet		
$f'_{ci} := 6.8$	ksi	$f_y := 60$	ksi
girder_size = "72W-inch"			
$w_{top\_flg} = 48$	inches	$w_{girder} = 0.953$	kips/ft
$t_{top\_flg\_min} = 3$	inches	$S_{bot} = -18825$	in^3
$t_{top\_flg\_max} = 5.5$	inches	$S_{top} = 17680$	in^3
$t_w = 6.5$	inches		

Lift point is assumed to be at the 1/10th point of the girder length.

E19-4.2 Lifting Stresses

Initial Girder Stresses (Taken from Prestressed Girder Output):

At the 1/10th Point, (positive values indicate compression)

$f_{i\_top\_0.1} := 0.284$ ksi

$f_{i\_bot\_0.1} := 3.479$ ksi

The initial stresses in the girder (listed above) are due to the prestressed strands and girder dead load moment. The girder dead load moment and resulting stresses are based on the girder being simply supported at the girder ends. These resulting stresses are subtracted from the total initial stresses to give the stresses resulting from the pressing force alone.

Moments and Shears due to the girder self weight:

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E27-1 DESIGN EXAMPLE - STEEL REINFORCED ELASTOMERIC BEARING

ThIs design example is for a 3-span prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.
(Example is current through LRFD Seventh Edition - 2015 Interim)

E27-1.1 Design Data

Bearing location: Abutment (Type A3)

Girder type: 72W

$L_{exp} := 220$ Expansion length, ft

$b_f := 2.5$ Bottom flange width, ft

$DL_{serv} := 167$ Service I limit state dead load, kips

$DL_{ws} := 23$ Service I limit state future wearing surface dead load, kips

$LL_{serv} := 62$ Service I limit state live load, kips

$h_{rcover} := 0.25$ Elastomer cover thickness, in

$h_s := 0.125$ Steel reinforcement thickness, in

$F_y := 36$ Minimum yield strength of the steel reinforcement, ksi

Temperature Zone:	C (Southern Wisconsin)	**LRFD [Fig. 14.7.5.2-1]**
Minimum Grade of Elastomer:	3	**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

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates were based on significant changes in the vehicle fleet, the emergence of many new barrier designs, increased interest in matching safety performance to levels of roadway utilization, new policies requiring the use of safety belts, and advances in computer simulation and other evaluation methods.

NCHRP Report 350 differs from NCHRP Report 230 in the following ways: it is presented in all-metric documentation, it provides a wider range of test procedures to permit safety performance evaluations for a wider range of barriers, it uses a pickup truck as the standard test vehicle in place of a passenger car, it defines other supplemental test vehicles, it includes a broader range of tests to provide a uniform basis for establishing warrants for the application of roadside safety hardware that consider the levels of use of the roadway facility, it includes guidelines for selection of the critical impact point for crash tests on redirecting-type safety hardware, it provides information related to enhanced measurement techniques related to occupant risk, and it reflects a critical review of methods and technologies for safety-performance evaluation.

In May of 1997, a memorandum from Dwight A. Horne, the FHWA Chief of the Federal-Aid and Design Division, on the subject of "Crash Testing of Bridge Railings" was published. This memorandum identified 68 crash-tested bridge rails, consolidated earlier listings, and established tentative equivalency ratings that related previous NCHRP Report 230 testing to NCHRP Report 350 test levels.

In 2009, AASHTO published the *Manual for Assessing Safety Hardware* (MASH). MASH is an update to, and supersedes, NCHRP Report 350 for the purposes of evaluating new safety hardware devices. AASHTO and FHWA jointly adopted an implementation plan for MASH that stated that all highway safety hardware accepted prior to the adoption of MASH – using criteria contained in NCHRP Report 350 – may remain in place and may continue to be manufactured and installed. In addition, highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. However, new highway safety hardware not previously evaluated must utilize MASH for testing and evaluation. MASH represents an update to crash testing requirements based primarily on changes in the vehicle fleet.

All bridge railings as detailed in the Wisconsin LRFD Bridge Standard Detail Drawings, with the exception of the type “F” steel railing, have been approved by FHWA per the crash tests as recommended in NCHRP Report 350. In order to use railings other than Bridge Office Standards, the railings must conform to MASH or must be crash tested rails which are available from the FHWA office. Any railings that are not crash tested must be reviewed by FHWA when they are used on a bridge, culvert, retaining wall, etc.

WisDOT policy states that railings that meet the criteria for Test Level 3 (TL-3) or greater shall be used on NHS roadways and all functional classes of Wisconsin structures (Interstate Highways, United States Highways, State Trunk Highways, County Trunk Highways, and Local Roadways) where the design speed exceeds 45 mph. Railings that meet Test Level 2 (TL-2) criteria may be used on non-NHS roadways where the design speed is 45 mph or less.

There may be unique situations that may require the use of a MASH or NCHRP Report 350 crash-tested railing of a different Test Level; a railing design using an older crash test methodology; or a modified railing system based on computer modeling, component testing, and or expert opinion. These unique situations will require an exception to be granted by the Bureau of Project Development and/or the Bureau of Structures. It is recommended that coordination of these unique situations occur early in the design process.

30.2 Railing Application

The primary purpose of bridge railings shall be to contain and redirect vehicles and/or pedestrians using the structure. In general, there are three types of bridge railings – Traffic Railings, Combination Railings, and Pedestrian Railings. The following guidelines indicate the typical application of each railing type:

1. Traffic Railings shall be used when a bridge is used exclusively for highway traffic.

   Traffic Railings can be composed of, but are not limited to: single slope concrete parapets, sloped face concrete parapets, vertical face concrete parapets, tubular steel railings, and timber railings.

2. Combination Railings can be used concurrently with a raised sidewalk on roadways with a design speed of 45 mph or less.

   Combination Railings can be composed of, but are not limited to: single slope concrete parapets with chain link fence, vertical face concrete parapets with tubular steel railings such as type 3T, and aesthetic concrete parapets with combination type C1-C6 railings.

3. Pedestrian Railings can be used at the outside edge of a bridge sidewalk when a Traffic Railing is used concurrently to separate highway and pedestrian traffic.

   Pedestrian Railings can be composed of, but are not limited to: chain link fence, tubular screening, vertical face concrete parapets with combination type C1-C6 or type 3T railings, and single slope concrete parapets.

See Figure 30.2-1 below for schematics of the three typical railing types.

Note that the railing types shown in Figure 30.2-1 shall be employed as minimums. At locations where a Traffic Railing is used at the traffic side of a sidewalk at grade, a Combination Railing may be used at the edge of deck in lieu of a Pedestrian Railing. At locations where a Combination Railing is used at the exterior edge of a raised sidewalk, a Traffic Railing may be used as an alternative as long as the requirements for Pedestrian Railings are met.

A “SS” or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and “HF” parapets shall be used as Traffic Railings for rehabilitation projects only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The “51F” parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. The vertical face parapet “A” can be used for all design speeds. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. Under some circumstances, the vertical face parapet “A” can be used as a Traffic Railing for design speeds exceeding 45 mph with the approval of the Bureau of Structures Development Section. The vertical face parapet “A” was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic railings may be used if crash tested according to Section 30.1 or follow the guidance provided in Section 30.4. See Chapter 4 – Aesthetics for CSS considerations.

   The Texas style aesthetic parapet, type “TX”, can be used as a Traffic/Pedestrian Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type “TX” parapet can be used. This parapet is very expensive; however, form liners simulating the openings can be used to reduce the cost of this parapet. The type “TX” parapet was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. The type “PF” tubular railing, as shown in the Standard Details, shall not be used on bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type “PF” railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type “C1” through “C6”, are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5” from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. Combination railing, type “3T”, without the recessed details on the parapet faces may be used when aesthetic details are not desired or when CSS funding is not available (see Chapter 4 – Aesthetics). These railings can only be used when the design

speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete parapets to which they are attached (i.e., if a type “C1” combination railing is attached to the top of a vertical face parapet type “A”, the parapet and railing combination meet crash test criteria for TL-4).

8. Chain Link Fence and Tubular Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between the roadway and a sidewalk at grade. Chain Link Fence, when attached to the top of a concrete Traffic Railing, can be used for design speeds exceeding 45 mph. Due to snagging and breakaway potential of the vertical spindles, Tubular Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets (“A” or “SS”) as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: “Type H (insert railing type) railing shall not be used”. The combination railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. 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.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated

for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing was crash-tested per NCHRP Report 350 and meets criteria for TL-4.

13. Type “NY3/NY4” steel railings, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “NY3/NY4” railings may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “NY3/NY4” railings also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. The type “NY4” railing may be used on a raised sidewalk where the design speed is 45 mph or less. However, the type “NY” railings are not allowed for use on prestressed box girder bridges. These railings shall be considered where the Region requests an open railing. The type “NY” railings were crash-tested per NCHRP Report 350 and meet criteria for TL-4.

14. The type "F" steel railing, as shown in the Standard Details, shall not 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.

15. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the *Facilities Development Manual (FDM) Standard Detail Drawings (SDD) 14b20*. Railing is not required on box culverts if the culvert is extended to provide an adequate clear zone as defined in *FDM 11-15-1*. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. The railing shall be provided only when it is cost effective as defined in *FDM Procedure 11-45-1*.

16. When the structure approach thrie beam is extended across the box culvert; refer to Standard Detail, Box Culvert Details for additional information. The minimum dimension between end of box and face of guard rail provides an acceptable rail deflection to prevent a vehicle wheel from traversing over the end of the box culvert. In almost every case, the timber posts with offset blocks and standard beam guard are used. Type "W" railing may be used for maintenance and box culvert extensions to mitigate the effect of structure modifications.

See the *FDM* for additional railing application requirements. See *11-45-1* and *11-45-2* for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See *11-35-1*, *Table 1.2* for requirements when barrier wall separation between roadway and sidewalk is necessary.

30.3 General Design Details

1. Epoxy coated bars are required for all concrete parapets, curbs, medians, and sidewalks.

2. Adhesive anchored parapets are allowed at interior Traffic Railing locations only when the adjacent exterior parapet is a crash test approved Traffic Railing per Section 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standard Details 30.10 and 30.14 for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in *FDM 11-45 Section 2.3.6.2.2* and *Section 2.3.6.2.3* respectively.

4. It is desirable to avoid attaching noise walls to bridge railings. However, in the event that noise walls are required to be located on bridge railings, compliance with the setback requirements stated in Section 30.4 and what is required in *FDM 11-45 Sections 2.3.6.2.2* and *2.3.6.2.3* is not required. *Note: WisDOT is currently investigating the future use of noise walls on bridge structures in Wisconsin.*

5. Temporary bridge barriers shall be designed in accordance with *FDM SDD 14b7*. Where temporary bridge barriers are being used for staged construction, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

6. Provide for expansion movement in tubular railings where expansion devices or concrete parapet deflection joints exist on the structure plan details. The tubular railing splice should be located over the joint and spaced evenly between railing posts. The tubular railing splice should be made continuous with a movable internal sleeve. If tubular railing is employed on conventional structures where expansion joints are likely to occur at the abutments only, the posts may be placed at equal spacings provided that no post is nearer than 2 feet from deflection joints in the parapet at the piers.

7. Refer to Standard Detail 30.07 – Vertical Face Parapet “A” – for detailing concrete parapet or sidewalk deflection joints. These joints are used based on previous experience with transverse deck cracking beneath the parapet joints.

8. Horizontal cracking has occurred in the past near the top of some concrete parapets which were slip formed. Similar cracking has not occurred on parapets cast in forms. Therefore, slip forming of bridge parapets shall not be allowed.

9. For beam guard type “W” railing, locate the expansion splice at a post or on either side of the expansion joint.

10. Sidewalks - If there is a Traffic Railing between the roadway and an at grade sidewalk, and the roadway side of the Traffic Railing is more than 11’-0” from the exterior edge of deck, access must be provided to the at grade sidewalk for the snooper truck to inspect the underside of the bridge. The sidewalk width must be

ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed as follows:

Minimum of 2'-3" behind the front face toe of the parapet when used with single slope parapets ("32SS", "36SS", "42SS", or "56SS").

Minimum of 2'-6" behind the front face toe of the parapet when used with sloped face parapets ("LF" or "HF").

Minimum of 2'-0" behind the front face of the parapet when used with vertical face parapets ("A").

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

4. Any concrete parapet placed directly on the deck may contain patterns or textures of any shape and length inset into the front face with the exception noted in #5. The maximum pattern or texture recess into the face of the barrier shall be ½". Note that the typical aesthetic formliner patterns shown in Standard Detail 4.01 are not acceptable for use on the front face of vehicle barriers.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings; especially in high speed applications where the aesthetic features will be negligible to the traveling public. In addition to the increased risk of vehicle snagging, aesthetic treatments on the front face of traffic railings are exposed to vehicle impacts, snowplow scrapes, and exposure to deicing chemicals. Due to these increased risks, future maintenance costs will increase.

5. No patterns with a repeating upward sloping edge or rim in the direction of vehicle traffic shall be permitted.

6. Staining should not be applied to the roadway side face of concrete traffic railings.

The application of aesthetics on bridge railings on structures in Wisconsin with a roadway design speed of 45 mph or less shall comply with the following guidance (see Chapter 4 – Aesthetics for CSS funding implications):

1. All Traffic Railings shall meet the crash testing guidelines outlined in Section 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed a minimum of 1'-0" behind the front face toe of the parapet.

3. Any railing placed on top of a concrete parapet shall be continuous over the full extents of the bridge.

4. Any concrete parapet placed directly on the deck or Combination Railing on a raised sidewalk may contain geometric patterns inset into the front face. The maximum recess into the face of the barrier shall be 1" and shall be placed concurrently with a

45° or flatter chamfered or beveled edge. See Standard Details 30.17 and 30.18 for one example of this type of aesthetic modification.

WisDOT highly recommends the use of smooth front faces of Traffic Railings and Combination Railings.

5. Any concrete parapet placed directly on the deck or Combination Railing on a raised sidewalk may contain textures of any shape and length inset into the front face. The maximum depth of the texture shall be ½". Note that the typical aesthetic formliner patterns shown in Standard Detail 4.01 are not acceptable for use on the front face of vehicle barriers.

   WisDOT highly recommends the use of smooth front faces of Traffic Railings and Combination Railings.

6. No patterns with a repeating upward sloping edge or rim in the direction of vehicle traffic shall be permitted.

7. Staining should not be applied to the roadway side face of concrete traffic railings. Staining is allowed on concrete surfaces of Combination Railings placed on a raised sidewalk.

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36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Currently, the *Bureau of Structures* does not require rating calculations for box culverts. See 45.8 for values to place on the plans for inventory and operating rating factors.

WisDOT Policy Item:

Current WisDOT policy is to not rate box culverts. In the future, rating requirements will be introduced as *AASHTO Manual for Bridge Evaluation (LRFR)* is updated to more thoroughly address box culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See section 45.6 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans. The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* does not thoroughly cover rating of concrete box culverts. See 45.8 for values to place on the plans for maximum (Wis-SPV) vehicle load.

36.2 General

Box culverts are reinforced concrete closed rigid frames which must support vertical earth and truck loads and lateral earth pressure. They may be either single or multi-cell. The most common usage is to carry water under roadways, but they are frequently used for pedestrian or cattle underpasses.

The minimum size for pedestrian underpasses is 8 feet high by 5 feet wide. The minimum size for cattle underpasses is 6 feet high by 5 feet wide. A minimum vertical opening of 5 feet is desirable for concrete box culverts for cleaning purposes.

Aluminum box culverts are not permitted by the Bureau of Structures.

Typical sections for the most frequently used box culverts are shown below.

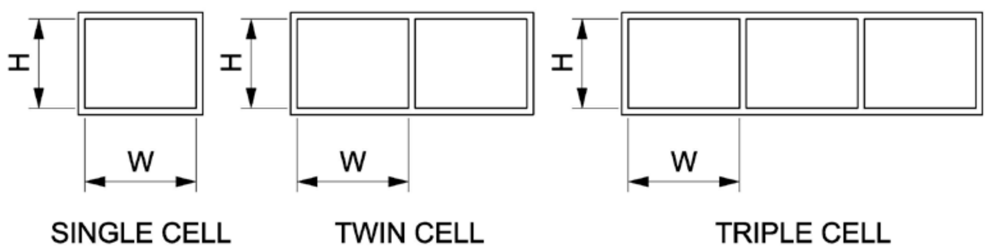

Figure 36.2-1
Typical Cross Sections

Hydraulic and other requirements at the site determine the required height and area of the box. Hydraulic design of box culverts is described in Chapter 8. Once the required height and area is determined, the selection of a single or multi-cell box is determined entirely from economics. Barrel lengths are computed to the nearest 6 inches. For multi-cell culverts the cell widths are kept equal.

36.2.1 Material Properties

The properties of materials used for concrete box culverts are as follows::

f'_c	=	specified compressive strength of concrete at 28 days, based on cylinder tests
	=	3.5 ksi for concrete in box culverts
f_y	=	60 ksi, specified minimum yield strength of reinforcement (Grade 60)
E_s	=	29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**
E_c	=	modulus of elasticity of concrete in box **LRFD [C5.4.2.4]**
	=	$(33{,}000)(K_1)(w_C)^{1.5}(f'_C)^{1/2}$ = 3586 ksi

Where:

K_1 = 1.0

W_C = 0.15 kcf, unit weight of concrete

n = Es / Ec = 8, modular ratio **LRFD [5.7.1]**

36.2.2 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Consider the hydraulics of the highway-stream crossing system in choosing the preferred design from the available alternatives. Estimates of life cycle costs and risks associated with each alternative help indicate which structure to select. Consider construction costs, maintenance costs, and risks of future costs to repair flood damage. Other considerations which may influence structure-type selection are listed in Table 36.2-1.

Bridges	
Advantages	Disadvantages
Less susceptible to clogging with drift, ice and debris	Require more structural maintenance than culverts
Waterway width increases with rising water surface until water begins to submerge structure	Piers and abutments susceptible to scour failure
Natural bottom for waterway	Susceptible to ice and frost forming on deck
Culverts	
Grade rises and widening projects sometimes can be accommodated by extending culvert ends	Silting in multiple barrel culverts may require periodic cleanout
Minimum structural maintenance	No increase in waterway area as stage rises above top of culvert
Usually easier and quicker to build than bridges	May clog with drift, debris or ice

Table 36.2-1
Advantages/Disadvantages of Structure Type

36.2.3 Staged Construction for Box Culverts

The inconvenience to the traveling public has often led to staged construction projects. Box culverts typically work well with staged construction. Any cell joint can be used for a staging joint. When the construction staging line cannot be determined in design to locate a cell joint, a contractor placed construction joint can be done with an extra set of dowel bars and the contractor field cutting the longitudinal bars.

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E36-1 Twin Cell Box Culvert LRFD

This example shows design calculations for a twin cell box culvert. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2015 Interim)***

E36-1.1 Design Criteria

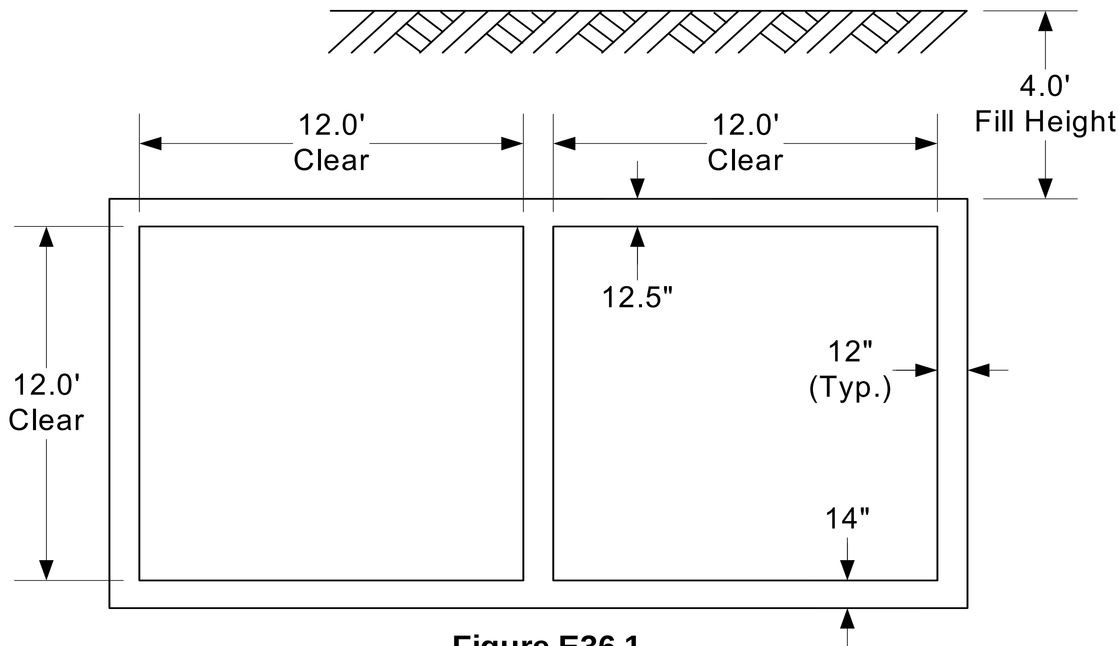


Figure E36.1
Box Culvert Dimensions

$NC = 2$	number of cells
$Ht = 12.0$	cell clear height, ft
$W_1 = 12.0$	cell 1 clear width, ft
$W_2 = 12.0$	cell 2 clear width, ft
$L = 134.0$	culvert length, ft
$t_{ts} = 12.5$	top slab thickness, in
$t_{bs} = 14.0$	bottom slab thickness, in
$t_{win} = 12.0$	interior wall thickness, in
$t_{wex} = 12.0$	exterior wall thickness, in
$H_{apron} := Ht + \frac{t_{ts}}{12}$	apron wall height above floor, ft

$H_{apron} = 13.04$ ft.

$f'_c := 3.5$	culvert concrete strength, ksi
$f_y := 60$	reinforcement yield strength, ksi
$E_s := 29000$	modulus of elasticity of steel, ksi
$skew = 0.0$	skew angle, degrees
$H_s = 4.00$	depth of backfill above top edge of top slab, ft
$w_c := 0.150$	weight of concrete, kcf
$cover_{bot} := 3$	concrete cover (bottom of bottom slab), in
$cover := 2$	concrete cover (all other applications), in
$LS_{ht} := 2.2$	live load surcharge height, ft (See Sect. 36.4.4)

Resistance factors, reinforced concrete cast-in-place box structures, **LRFD [Table 12.5.5-1]**

$\phi_f := 0.9$	resistance factor for flexure
$\phi_v := 0.85$	resistance factor for shear

Calculate the span lengths for each cell (measured between centerlines of walls)

$$S_1 := W_1 + \frac{1}{12}\left(\frac{t_{win}}{2} + \frac{t_{wex}}{2}\right) \qquad \boxed{S_1 = 13.00} \text{ ft}$$

$$S_2 = W_2 + \frac{1}{12}\left(\frac{t_{wex}}{2} + \frac{t_{win}}{2}\right) \qquad \boxed{S_2 = 13.00} \text{ ft}$$

Verify that the box culvert dimensions fall within WisDOT's minimum dimension criteria. Per Sect. 36.2, the minimum size for pedestrian underpasses is 8 feet high by 5 feet wide. The minimum size for cattle underpass is 6 feet high by 5 feet wide. A minimum height of 5 feet is desirable for cleanout purposes.

Does the culvert meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Verify that the slab and wall thicknesses fall within WisDOT's minimum dimension criteria. Per Sect. 36.5, the minimum thickness of the top and bottom slab is 6.5 inches. Per Sect. 36.5 [Table 36.5-1], the minimum wall thickness varies with respect to cell height and apron wall height.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

Do the slab and wall thicknesses meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Since this example has more than 2.0 feet of fill, edge beams are not req'd, **LRFD [C12.11.2.1]**

E36-1.2 Modulus of Elasticity of Concrete Material

Per Sect. 36.2.1, use $f`_c$ = 3.5 ksi for culverts. Calculate value of E_C per **LRFD [C5.4.2.4]**:

$\boxed{K_1 := 1}$ $E_{c\_calc} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ $\boxed{E_{c\_calc} = 3586.616}$ ksi

$E_c := 3600$ ksi modulus of elasticity of concrete, per Sect. 9.2

E36-1.3 Loads

$\gamma_s := 0.120$ unit weight of soil, kcf

Per Sect. 36.5, a haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Minimum haunch depth and length is 6 inches. Haunch depth is increased in 3 inch increments. For the first iteration, assume there are no haunches.

$h_{hau} := 0.0$ haunch height, in

$l_{hau} := 0.0$ haunch length, in

$wt_{hau} = 0.0$ weight of one haunch, kip

E36-1.3.1 Dead Loads

Dead Load (DC):

top slab dead load:

$$w_{dlts} := w_c \cdot \frac{t_{ts}}{12} \cdot 1$$ $\boxed{w_{dlts} = 0.156}$ klf

bottom slab dead load:

$$w_{dlbs} := w_c \cdot \frac{t_{bs}}{12} \cdot 1$$ $\boxed{w_{dlbs} = 0.175}$ klf

Wearing Surface (DW):

Per Sect. 36.4.2, the weight of the future wearing surface is zero if there is any fill depth over the culvert. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 0.020 ksf.

$w_{ws} = 0.000$ weight of future wearing surface, ksf

Vertical Earth Load (EV):

Calculate the modification of earth loads for soil-structure interaction per **LRFD [12.11.2.2]**. Per the policy item in Sect. 36.4.3, embankment installations are always assumed.

Installation_Type = "Embankment"

$\gamma_s = 0.120$ unit weight of soil, kcf

$B_c = 27.00$ outside width of culvert, ft
(measured between outside faces of exterior walls)

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

Calculate the soil-structure interaction factor for embankment installations:

$$F_e := 1 + 0.20 \cdot \frac{H_s}{B_c}$$ $\boxed{F_e = 1.03}$

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section:

$\boxed{F_e = 1.03}$

Calculate the total unfactored earth load:

$W_E := F_e \cdot \gamma_s \cdot B_c \cdot H_s$ $\boxed{W_E = 13.34}$ klf

Distrubute the total unfactored earth load to be evenly distributed across the top of the culvert:

$w_{sv} := \frac{W_E}{B_c}$ $\boxed{w_{sv} = 0.494}$

Horizontal Earth Load (EH):

Soil horizontal earth load (magnitude at bottom and top of wall): **LRFD [3.11.5.1]**

$k_o := 0.5$ coefficient of at rest lateral earth pressure per Sect. 36.4.3

$\gamma_s = 0.120$ unit weight of soil, kcf

$w_{sh\_bot} := k_o \cdot \gamma_s \cdot \left(Ht + \frac{t_{ts}}{12} + \frac{t_{bs}}{12} + H_s\right) \cdot 1$ $\boxed{w_{sh\_bot} = 1.09}$ klf

$w_{sh\_top} := k_o \cdot \gamma_s \cdot (H_s) \cdot 1$ $\boxed{w_{sh\_top} = 0.24}$ klf

Live Load Surcharge (LS):

Soil live load surcharge: **LRFD [3.11.6.4]**

$k_o = 0.5$ coefficient of lateral earth pressure

$\gamma_s = 0.120$ unit weight of soil, kcf

$LS_{ht} = 2.2$ live load surcharge height per Sect. 36.4.4, ft

$w_{sll} := k_o \cdot \gamma_s \cdot LS_{ht} \cdot 1$ $\boxed{w_{sll} = 0.13}$ klf

E36-1.3.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = design truck (no lane) **LRFD [3.6.1.3.3]**

design tandem (no lane)

For the Wisconsin Standard Permit Vehicle (Wis-SPV) Check:

The Wis-SPV vehicle is to be checked during the design phase to make sure it can carry a minimum vehicle load of 190 kips. See Section 36.1.3 of the Bridge Manual for requirements pertaining to the Wis-SPV vehicle check.

E36-1.4 Live Load Distribution

Live loads are distributed over an equivalent area, with distribution components both parallel and perpendicular to the span, as calculated below. Per **LRFD [3.6.1.3.3]**, the live loads to be placed on these widths are axle loads (i.e., two lines of wheels) without the lane load. The equivalent distribution width applies for both live load moment and shear.

E36-1.5 Equivalent Strip Widths for Box Culverts

The calculations for depths of fill less than 2.0 ft, per **LRFD [4.6.2.10]** are not required for this example. The calcuations are shown for illustration purposes only.

The calculations below follow **LRFD [4.6.2.10.2]** - Case 1: Traffic Travels Parallel to Span. If traffic travels perpendicular to the span, follow **LRFD [4.6.2.10.3]** - Case 2: Traffic Travels Perpendicular to Span, which states to follow **LRFD [4.6.2.1]**.

Per **LRFD [4.6.2.10.2]**, when traffic travels primarily parallel to the span, culverts shall be analyzed for a single loaded lane with a single lane multiple presence factor (mpf).

Therefore, $mpf = 1.2$

Perpendicular to the span:

It is conservative to use the largest distribution factor from each span of the structure across the entire length of the culvert. Therefore, use the smallest span to calculate the smallest strip width. That strip width will provide the largest distribution factor.

$S := \min\left(W_1, W_2\right)$ clear span, ft $\boxed{S = 12.00}$ ft

The equivalent distribution width perpendicular to the span is:

$E_{perp} := \frac{1}{12}\cdot(96 + 1.44\cdot S)$ $\boxed{E_{perp} = 9.44}$ ft

Parallel to the span:

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

$L_T := 10$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$LLDF = 1.15$ live load distribution factor. From **LRFD [4.6.2.10.2]**, LLDF = 1.15 as specified in **LRFD [Table 3.6.1.2.6a-1]** for select granular backfill

The equivalent distribution width parallel to the span is:

$E_{parallel} := \frac{1}{12}\cdot\left(L_T + LLDF\cdot H_s\cdot 12\right)$ $\boxed{E_{parallel} = 5.43}$ ft

The equivalent distribution widths parallel and perpendicular to the span create an area that the axial load shall be distributed over. The equivalent area is:

$E_{area} := E_{perp}\cdot E_{parallel}$ $\boxed{E_{area} = 51.29}$ ft^2

For depths of fill 2.0 ft. or greater calculate the size of the rectangular area that the wheels are considered to be uniformly distributed over, per Sect. 36.4.6.2.

$L_T = 10.0$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$W_T := 20$ width of tire contact area, in **LRFD [3.6.1.2.5]**

The length and width of the equivalent area for 1 wheel are: **LRFD [3.6.1.2.6b]**

$L_{eq\_i} := L_T + LLDF \cdot H_s \cdot 12$ $\boxed{L_{eq\_i} = 65.20}$ in

$W_{eq\_i} := W_T + LLDF \cdot H_s \cdot 12 + 0.06 \cdot \max(W_1, W_2)\, 12$ $\boxed{W_{eq\_i} = 83.84}$ in

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area, **LRFD [3.6.1.2.6a]**.

Check if the areas overlap = "Yes, the areas overlap" therefore, use the following length and width values for the equivalent area for 1 wheel:

	Front and Rear Wheels:		Center Wheel:	
Length	$L_{eq13} = 65.2$	in	$L_{eq2} = 65.2$	in
Width	$W_{eq13} = 77.9$	in	$W_{eq2} = 77.9$	in
Area	$A_{eq13} = 5080.4$	in^2	$A_{eq2} = 5080.4$	in^2

Per **LRFD [3.6.1.2.2]**, the weights of the design truck wheels are below. (Note that one axle load is equal to two wheel loads.)

$W_{wheel1i} := 4000$ front wheel weight, lbs

$W_{wheel23i} := 16000$ center and rear wheel weights, lbs

The effect of single and multiple lanes shall be considered. For this problem, a single lane with the single lane multiple presence factor (mpf) governs. Applying the single lane multiple presence factor:

$W_{wheel1} := mpf \cdot W_{wheel1i}$ $W_{wheel1} = 4800.00$ lbs $mpf = 1.20$

$W_{wheel23} := mpf \cdot W_{wheel23i}$ $W_{wheel23} = 19200.00$ lbs

For single-span culverts, the effects of the live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects of the live load may be neglected where the depth of fill exceeds the distance between faces of endwalls, **LRFD [3.6.1.2.6a]**.
Note: The wheel pressure values shown here are for the 14'-0" variable axle spacing of the design truck, which controls over the design tandem for this example. In general, all variable axle spacings of the design truck and the design tandem must be investigated to account for the maximum response. Dividing the wheel loads (incl. mpf) by the equivalent area gives:

$LL1 = 0.94$ live load pressure (front wheel), psi

$LL2 = 3.78$ live load pressure (center wheel), psi

$LL3 = 3.78$ live load pressure (rear wheel), psi

E36-1.6 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in Chapter 36 of this manual and as indicated below.

E36-1.6.1 Load Factors

From **LRFD [Table 3.4.1-1]** and **LRFD [Table 3.4.1-2]**:

Per the policy item in Sect. 36.4.3: Assume box culverts are closed, rigid frames for Strength 1 (EV-factor). Assume active earth pressure to be conservative for Strength 1 (EH-factor).

	Strength 1	Service 1
DC	$\gamma st_{DCmax} := 1.25$	$\gamma s1_{DC} := 1.0$
	$\gamma st_{DCmin} := 0.9$	
DW	$\gamma st_{DWmax} := 1.5$	$\gamma s1_{DW} := 1.0$
	$\gamma st_{DWmin} := 0.65$	
EV	$\gamma st_{EVmax} := 1.35$	$\gamma s1_{EV} := 1.0$
	$\gamma st_{EVmin} := 0.9$	
EH	$\gamma st_{EHmax} := 1.50$	$\gamma s1_{EH} := 1.0$
	$\gamma st_{EHmin} := 0.5$ **LRFD [3.11.7]**	
LS	$\gamma st_{LSmax} := 1.75$	$\gamma s1_{LS} := 1.0$
	$\gamma st_{LSmin} := 0$	
LL	$\gamma st_{LL} := 1.75$	$\gamma s1_{LL} := 1.0$

Dynamic Load Allowance (IM) is applied to the truck and tandem. From **LRFD [3.6.2.2]**, IM of buried components varies with depth of cover above the structure and is calculated as:

$IM := 33 \cdot (1.0 - 0.125 \cdot H_S)$ (where H_S is in feet) $IM = 16.50$

If IM is less than 0, use IM = 0 $\boxed{IM = 16.50}$

E36-1.6.2 Dead Load Moments and Shears

The unfactored dead load moments and shears for each component are listed below (values are per 1-foot width and are in kip-ft and kip, respectively):

Exterior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-1.52	-1.44	-5.14	-1.01	0.00
0.1	-1.42	-1.54	-0.12	-0.14	0.00
0.2	-1.31	-1.63	3.53	0.55	0.00
0.3	-1.21	-1.73	5.92	1.04	0.00
0.4	-1.10	-1.82	7.14	1.34	0.00
0.5	-1.00	-1.91	7.30	1.46	0.00
0.6	-0.89	-2.01	6.51	1.38	0.00
0.7	-0.79	-2.10	4.87	1.12	0.00
0.8	-0.68	-2.19	2.49	0.66	0.00
0.9	-0.58	-2.29	-0.54	0.01	0.00
1.0	-0.48	-2.38	-4.11	-0.82	0.00

Interior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

E36-1.7 Design Reinforcement Bars

Design of the corner bars is illustrated below. Calculations for bars in other locations are similar.

Design Criteria:

For corner bars, use the controlling thickness between the slab and wall. The height of the concrete design section is:

$$h := \min(t_{ts}, t_{bs}, t_{wex})$$ $\boxed{h = 12.00}$ in

Use a 1'-0" design width:

$b := 12.0$ width of the concrete design section, in

$cover = 2.0$ concrete cover, in Note: The calculations here use 2" cover for the top slab and walls. Use 3" cover for the bottom of the bottom slab (not shown here).

$Mstr1_{CB} = 17.34$ design strength moment, kip-ft

$Ms1_{CB} = 11.18$ design service moment, kip-ft

$f_s := f_y$ reinforcement yield strength, ksi $f_y = 60.00$ ksi

$Bar_{No} := 5$ assume #5 bars (for d_s calculation)

$Bar_D(Bar_{No}) = 0.63$ bar diameter, in

Calculate the estimated distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement. **LRFD [5.7.3.2.2]**

$$d_{s\_i} := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_{s\_i} = 9.69}$ 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:

$$R_n := \frac{Mstr1_{CB} \cdot 12}{\phi_f \cdot b \cdot d_{s\_i}^{\,2}}$$ $\boxed{R_n = 0.21}$ ksi

Calculate the reinforcement ratio:

$$\rho := 0.85 \cdot \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1.0 - \frac{2 \cdot R_n}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.0035}$

Calculate the required area of steel:

$A_{s\_req'd} := \rho \cdot b \cdot d_{s\_i}$ $\boxed{A_{s\_req'd} = 0.41}$ in^2

Given the required area of steel of $A_{s\_req'd} = 0.41$, try #5 bars at 7.5" spacing:

$Bar_{No} := 5$ bar size

$spacing := 7.5$ bar spacing, in

The area of one reinforcing bar is:

$A_{s\_1bar} := Bar_A(Bar_{No})$ $\boxed{A_{s\_1bar} = 0.31}$ in^2

Calculate the area of steel in a 1'-0" width

$$A_s := \frac{A_{s\_1bar}}{\frac{spacing}{12}}$$ $\boxed{A_s = 0.50}$ in^2

Check that the area of steel provided is larger than the required area of steel

Is $A_s = 0.50\ in^2 \geq A_{s\_req'd} = 0.41\ in^2$ $\boxed{check = \text{"OK"}}$

Recalculate d_c and d_s based on the actual bar size used.

$$d_c := cover + \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_c = 2.31}$ in

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_s = 9.69}$ in

Per **LRFD [5.7.2.2]**, The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65. The factor α_1 shall be taken as 0.85 for concrete strength not exceeding 10.0 ksi.

$\boxed{\beta_1 = 0.85}$ $\boxed{\alpha_1 = 0.85}$

Per **LRFD [5.7.2.1]**, if $\frac{c}{d_s} \leq 0.6$ (for f_y = 60 ksi) then reinforcement has yielded and the assumption is correct.

"c" is defined as the distance between the neutral axis and the compression face (inches).

$$c := \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$ $\boxed{c = 0.98}$ in

Check that the reinforcement will yield:

Is $\frac{c}{d_s} = 0.10 \leq 0.6$? $\boxed{check = \text{"OK"}}$

therefore, the reinforcement will yield

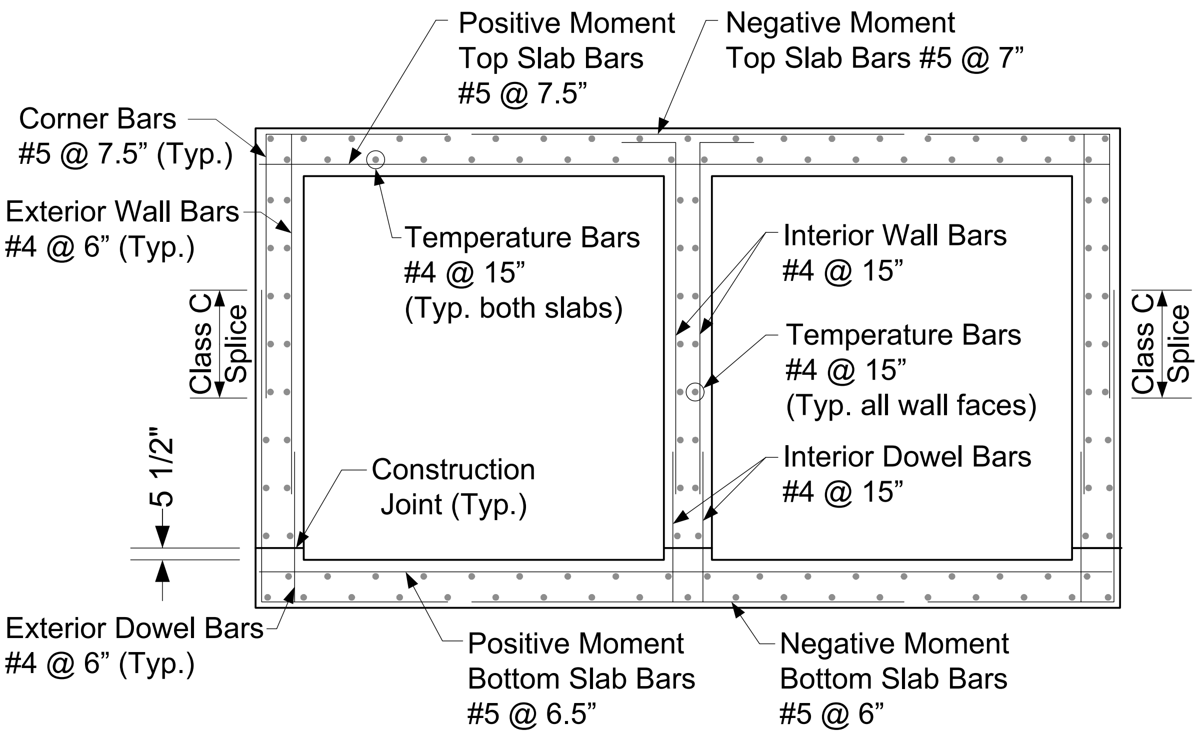
E36-1.9 Distribution Reinforcement

Per **LRFD [9.7.3.2]**, reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

Distribution steel is not required when the depth of fill over the slab exceeds 2 feet, **LRFD [5.14.4.1**].

E36-1.10 Reinforcement Details

The reinforcement bar size and spacing required from the strength and serviceability calcuations above are shown below:



E36-1.11 Cutoff Locations

Determine the cutoff locations for the corner bars. Per Sect. 36.6.1, the distance "L" is computed from the maximum negative moment envelope for the top slab.

The cutoff lengths are in feet, measured from the inside face of the exterior wall.

Initial Cutoff Locations:

The initial cutoff locations are determined from the inflection points of the moment diagrams.

Corner Bars	$CutOff1_{CBH\_i} = 2.64$	$CutOff2_{CBH\_i} = 1.57$	Horizontal
		$CutOff2_{CBV\_i} = 2.37$	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS\_i} = 1.26$	$CutOff2_{PTS\_i} = 1.86$	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS\_i} = 1.27$	$CutOff2_{PBS\_i} = 1.97$	
Negative Moment Top Slab Bars	$CutOff1_{NTS\_i} = 8.63$	$CutOff2_{NTS\_i} = 10.32$	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS\_i} = 8.97$	$CutOff2_{NBS\_i} = 10.56$	

For the second cutoff location for each component, the following checks shall be completed:

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]**:

The required capacity at the second cutoff location (for the vertical leg of the corner bar):

$Mstr1_{CBV2} = 7.89$ strength moment at the second cutoff location, kip-ft

The usable capacity of the remaining bars is calculated as follows:

$$A_{s2} := \frac{A_s}{2} \qquad \boxed{A_{s2} = 0.25}\ \text{in}^2$$

$$c2 := \frac{A_{s2} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \qquad \boxed{\beta_1 = 0.85} \quad \boxed{\alpha_1 = 0.85} \qquad \boxed{c2 = 0.49}\ \text{in}$$

$$a2 := \beta_1 \cdot c2 \qquad \boxed{a2 = 0.42}\ \text{in}$$

$$M_{n2} := \left[A_{s2} \cdot f_s \cdot \left(d_s - \frac{a2}{2}\right) \frac{1}{12}\right] \qquad \boxed{M_{n2} = 11.8}\ \text{kip-ft}$$

$$M_{r2} := \phi_f \cdot M_{n2} \qquad \boxed{M_{r2} = 10.6}\ \text{kip-ft}$$

Is $M_{r2} = 10.6$ kip-ft greater than the lesser of M_{cr} and $1.33{*}M_{str}$? $\boxed{check = \text{"OK"}}$

$\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CBV2} = 10.5}$ kip-ft

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi).

$Ms1_{CBV2} = 3.43$ service moment at the second cutoff location, kip-ft

$$f_{ss2} := \frac{Ms1_{CBV2} \cdot 12}{A_{s2} \cdot (j) \cdot (h - d_c)}$$ $\boxed{f_{ss2} = 18.54}$ ksi

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max2\_1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss2}} - 2 \cdot d_c$$ $s_{max2\_1} = 23.53$ in

$s_{max2\_2} := s_{max2}$ $s_{max2\_2} = 18.00$ in

$s_{max} := \min(s_{max2\_1}, s_{max2\_2})$ $\boxed{s_{max} = 18.00}$ in

Check that the provided spacing (for half of the bars) is less than the maximum allowable spacing

$spacing2 := 2 \cdot spacing$ $spacing2 = 15.00$ in

Is $spacing2 = 15.00$ in $\leq$ $s_{max} = 18.00$ in $\boxed{check = \text{"OK"}}$

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 [5.11.1.2.1]**:

$$MaxDepth := \max\left(t_{ts} - cover, t_{wex} - cover, t_{bs} - cover_{bot}\right) \qquad MaxDepth = 11.00 \quad in$$

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

15 x bar diameter $\dfrac{15 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.78$ ft

1/20 times clear span $\dfrac{\max\left(W_1, W_2\right)}{20} = 0.60$ ft

The maximum of the values listed above:

$$ExtendLength\_gen_{CB} = 0.89 \quad ft$$

Extension lengths for negative moment reinforcement per **LRFD [5.11.1.2.3]**:

Effective member depth $\dfrac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.89$ ft

12 x bar diameter $\dfrac{12 \cdot Bar_D\left(Bar_{No\_CB}\right)}{12} = 0.63$ ft

0.0625 times clear span $0.0625 \max\left(W_1, W_2\right) = 0.75$ ft

The maximum of the values listed above:

$$ExtendLength\_neg_{CB} = 0.89 \quad ft$$

The development length:

$$DevLength_{CB} = 1.00 \quad ft$$

E36-1.12 Shear Analysis

Analyze walls and slabs for shear

E36-1.12.1 Factored Shears

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored shears for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Shears

$$V_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot V_{DC} + \gamma st_{DW} \cdot V_{DW} + \gamma st_{EV} \cdot V_{EV} + \gamma st_{EH} \cdot V_{EH} + \gamma st_{LS} \cdot V_{LS} + \gamma st_{LL} \cdot V_{LL} \right)$$

Exterior Wall	$Vstr1_{XW} = 8.69$	kip
Interior Wall	$Vstr1_{IW} = 0.40$	kip
Top Slab	$Vstr1_{TS} = 12.20$	kip
Bottom Slab	$Vstr1_{BS} = 12.16$	kip

Service 1 Shears

$$V_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot V_{DC} + \gamma s1_{DW} \cdot V_{DW} + \gamma s1_{EV} \cdot V_{EV} + \gamma s1_{EH} \cdot V_{EH} + \gamma s1_{LS} \cdot V_{LS} + \gamma s1_{LL} \cdot V_{LL} \right)$$

Exterior Wall	$Vs1_{XW} = 5.64$	kip
Interior Wall	$Vs1_{IW} = 0.23$	kip
Top Slab	$Vs1_{TS} = 7.62$	kip
Bottom Slab	$Vs1_{BS} = 7.96$	kip

E36-1.12.2 Concrete Shear Resistance

Check that the nominal shear resistance, V_n, of the concrete in the top slab is adequate for shear without shear reinforcement per **LRFD [5.14.5.3]**.

$$V_n = V_c = \left(0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u} \right) \cdot b \cdot d_s \le 0.126 \cdot \sqrt{f'_c} \cdot b \cdot d_s$$

$f'_c = 3.5$ culvert concrete strength, ksi

$A_{s\_TS} = 0.53$ area of reinforcing steel in the design width, in^2/ft width

$h := t_{ts}$ height of concrete design section, in $h = 12.50$ in

Calculate d_s, the distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 10.19} \text{ in}$$

$$V_u := Vstr1_{TS} \qquad \boxed{V_u = 12.2} \text{ kips}$$

$M_u = 264.01$ factored moment occurring simultaneously with V_u, kip-in

$b := 12$ design width, in

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$\frac{V_u \cdot d_s}{M_u}$ shall not be taken to be greater than 1.0 $\qquad \frac{V_u \cdot d_s}{M_u} = 0.47$ < 1.0 OK

$$V_{r1s} := \phi_v \cdot \left[\left(0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_{s\_TS}}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s\right] \qquad \boxed{V_{r1s} = 14.1} \text{ kips}$$

but ≤ $V_{r2s} := \phi_v \cdot \left(0.126 \cdot \sqrt{f'_c} \cdot b \cdot d_s\right)$ $\qquad \boxed{V_{r2s} = 24.5}$ kips

$$V_{rs} := \min(V_{r1s}, V_{r2s}) \qquad \boxed{V_{rs} = 14.1} \text{ kips}$$

Check that the provided shear capacity is adequate:

Is $V_u = 12.2$ kip ≤ $V_{rs} = 14.1$ kip ? $\qquad \boxed{check = \text{"OK"}}$

Note: For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than: **LRFD[5.14.5.3]** $\qquad 0.0948 \cdot \sqrt{f'_c}\ b \cdot d_s$

V_c for slabs simply supported need not be taken to be less than: $\qquad 0.0791 \cdot \sqrt{f'_c}\ b \cdot d_s$

LRFD [5.8] and **LRFD [5.13.3.6]** apply to slabs of box culverts with less than 2.0 ft of fill.

Check that the nominal shear resistance, V_n, of the concrete in the walls is adequate for shear without shear reinforcement per **LRFD [5.8.3.3]**. Calculations shown are for the exterior wall.

$$V_n = V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v \le 0.25 \cdot f'_c \cdot b_v \cdot d_v$$

$\beta := 2$ **LRFD [5.8.3.4.1]**

$f'_c = 3.5$ culvert concrete strength, ksi

$b_v := 12$ effective width, in

$h := t_{wex}$ height of concrete design section, in $\qquad h = 12.00$ in

Distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2} \qquad \boxed{d_s = 9.69} \text{ in}$$

The effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; **LRFD [5.8.2.9]**

$$d_{v\_i} = d_s - \frac{a}{2}$$

from earlier calculations:

$$\boxed{\beta_1 = 0.85}$$

$$\boxed{f_s = 60} \text{ ksi}$$

$$\boxed{A_{s\_XW} = 0.40} \text{ in}^2$$

The distance between the neutral axis and the compression face:

$$c := \frac{A_{s\_XW} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_v} \qquad \boxed{\beta_1 = 0.85} \quad \boxed{\alpha_1 = 0.85} \qquad \boxed{c = 0.79} \text{ in}$$

$$a := \beta_1 \cdot c \qquad \boxed{a = 0.67} \text{ in}$$

The effective shear depth:

$$d_{v\_i} := \left(d_s - \frac{a}{2}\right) \qquad \boxed{d_{v\_i} = 9.35}$$

d_v need not be taken to be less than the greater of 0.9 d_s or 0.72h (in.)

$$d_v := \max\left(d_{v\_i}, \max\left(0.9d_s, 0.72t_{wex}\right)\right) \qquad 0.9 \cdot d_s = 8.72$$

$$d_v = 9.35 \text{ in} \qquad 0.72 \cdot t_{wex} = 8.64$$

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**.

Therefore the usable capacity is:

$$V_{r1w} := \phi_v \cdot \left(0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r1w} = 11} \text{ kips}$$

$$\text{but} \le \quad V_{r2w} := \phi_v \cdot \left(0.25 \cdot f'_c \cdot b_v \cdot d_v\right) \qquad \boxed{V_{r2w} = 83} \text{ kips}$$

$$V_{rw} := \min\left(V_{r1w}, V_{r2w}\right) \qquad \boxed{V_{rw} = 11} \text{ kips}$$

$$V_u := Vstr1_{XW} \qquad \boxed{V_u = 8.7} \text{ kips}$$

Check that the provided shear capacity is adequate:

Is $V_u = 8.7$ kip $\leq$ $V_{rw} = 11.3$ kip ? $\boxed{\text{check} = \text{"OK"}}$

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37.1 Structure Selection

Most pedestrian bridges are located in urban areas and carry pedestrian and/or bicycle traffic over divided highways, expressways and freeway systems. The structure type selected is made on the basis of aesthetics and economic considerations. A wide variety of structure types are available and each type is defined by the superstructure used. Some of the more common types are as follows:

- Concrete Slab
- Prestressed Concrete Girder
- Steel Girder
- Prefabricated Truss

Several pedestrian bridges are a combination of two structure types such as a concrete slab approach span and steel girder center spans. One of the more unique pedestrian structures in Wisconsin is a cable stayed bridge. This structure was built in 1970 over USH 41 in Menomonee Falls. It is the first known cable stayed bridge constructed in the United States. Generally, pedestrian bridges provide the designer the opportunity to employ long spans and medium depth sections to achieve a graceful structure.

Pedestrian boardwalks will not be considered “bridges” when their clear spans are less than or equal to 20 feet, and their height above ground and/or water is less than 10 feet. Boardwalks falling under these constraints will not be required to follow the design requirements in the WisDOT Bridge Manual, but will need to follow the standards established in the *Wisconsin Bicycle Facility Design Handbook.*

37.2 Specifications and Standards

The designer shall refer to the following related specifications:

- "*AASHTO LRFD Bridge Design Specifications*"
- *"AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges",* hereafter referred to as the *"Pedestrian Bridge Guide"*
- See *Standardized Special Provision (STSP)* titled "*Prefabricated Steel Truss Pedestrian Bridge LRFD*" for the requirements for this bridge type

For additional design information, refer to the appropriate Wisconsin Bridge Manual chapters relative to the structure type selected.

The pedestrian live load (PL) shall be as follows: (from "*Pedestrian Bridge Guide")*

- 90 psf **[Article 3.1]**
- Dynamic load allowance is not applied to pedestrian live loads **[Article 3.1]**

The vehicle live load shall be applied as follows: (from *"Pedestrian Bridge Guide")*

- Design for an occasional single maintenance vehicle live load (LL) **[Article 3.2]**

Clear Bridge Width (w)	Maintenance Vehicle
7 ft $\leq$ w $\leq$ 10 ft	H5 Truck (10,000 lbs)
w > 10 ft	H10 Truck (20,000 lbs)

- Clear bridge widths of less than 7 feet need not be designed for maintenance vehicles. **[Article 3.2]**
- The maintenance vehicle live load shall not be placed in combination with the pedestrian live load. **[Article 3.2]**
- Dynamic load allowance is not applied to the maintenance vehicle. **[Article 3.2]**
- Strength I Limit State shall be used for the maintenance vehicle loading. **[Article 3.2, 3.7]**

On Federal Aid Structures FHWA requests a limiting gradient of 8.33 percent (1:12) on ramps for pedestrian facilities to accommodate the physically handicapped and elderly as recommended by the "*American Standard Specifications for Making Buildings and Other Facilities Accessible to, and Usable by, the Physically Handicapped*". This is slightly flatter than the gradient guidelines set by AASHTO which states gradients on ramps should not be more than 15 percent and preferably not steeper than 10 percent.

The minimum inside clear width of a pedestrian bridge on a pedestrian accessible route is 8 feet. (*AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, 2004).*

The width required is based on the type, volume, and direction of pedestrian and/or bicycle traffic.

The vertical clearance on the pedestrian bridge shall be a minimum of 10 feet for bicyclists' comfort and to allow access for maintenance and emergency vehicles. The Wisconsin Department of Natural Resources recommends a vertical clearance on the bridge of at least 12 feet to accommodate maintenance and snow grooming equipment on state trails. Before beginning the design of the structure, the Department of Natural Resources and the Bureau of Structures should be contacted for the vertical clearance requirements for all vehicles that require access to the bridge.

In addition, ramps should have rest areas or landings 5 feet to 6 feet in length which are level and safe. Rest area landings are mandatory when the ramp gradient exceeds 5 percent. Recommendations are that landings be spaced at 60 foot maximum intervals, as well as wherever a ramp turns. This value is based on a maximum gradient of 8.33 percent on pedestrian ramps, and placing a landing at every 5 feet change in vertical elevation. Also, ramps are required to have handrails on both sides. See Standard Details for handrail location and details.

Minimum vertical clearance for a pedestrian overpass can be found in the *Facilities Development Manual (FDM)* Procedure 11-35-1, Attachment 1.8 and 1.9. Horizontal clearance is provided in accordance with the requirement found in *(FDM)* Procedure 11-35-1, Attachment 1.5 and 1.6.

Live load deflection limits shall be in accordance with the provisions of **LRFD [2.5.2.6.2]** for the appropriate structure type.

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spalling of concrete surfaces and exposure of reinforcing steel, or disintegration of masonry jointing.

- Dolphins - Where depth of water and other conditions are suitable, the driving of pile clusters may be considered. Such clusters have the piles lashed together with cable to promote integral action. The clusters should be flexible to be effective in absorbing impact through deflection.

- Cellular dolphins - May be filled with concrete, loose materials or materials suitable for grouting.

- Floating shear booms - Where the depth of water or other conditions precludes the consideration of dolphins or integral pier protection, floating sheer booms may be used. These are suitably shaped and positioned to protect the pier and are anchored to allow deflection and absorption of energy. Anchorage systems should allow for fluctuations in water level due to stream flow or tidal action.

- Hydraulic devices - Such as suspended cylinders engaging a mass of water to absorb or deflect the impact energy may be used under certain conditions of water depth or intensity of impact. Such cylinders may be suspended from independent caissons, booms projecting from the pier or other supports. Such devices are customarily most effective in locations subject to little fluctuations of water levels.

- Fender systems - Constructed using piling with horizontal wales, is a common means of protection where water depth is not excessive and severe impacts are not anticipated.

- Other types of various protective systems have been successfully used and may be considered by the Engineer. Criteria for the design of protective systems cannot be specified to be applicable to all situations. Investigation of local conditions is required in each case, the results of which may then be used to apply engineering judgment to arrive at a reasonable solution.

38.4 Overpass Structures

Highway overpass structures are placed when the incidences of train and vehicle crossings exceeds certain values specified in the *Facilities Development Manual* (*FDM*). The separation provides a safer environment for both trains and vehicles.

In preparing the preliminary plan which will be sent to the railroad company for review and approval several items of data must be determined.

- Track Profile - In order to maintain clearances under existing structures when the track was upgraded with new ballast, the railroad company did not change the track elevation under the structure causing a sag in the gradeline. The track profile would be raised with a new structure and the vertical clearance for the structure should consider this.

- Drainage - Hydraulic analysis is required if any excess drainage will occur along the rail line or into existing drainage structures. Deck drains shall not discharge onto railroad track beds.

- Horizontal Clearances - The railroad system is expanding just as the highway system. Contact the railroad company for information about adding another track or adding a switching yard under the proposed structure.

- Safety Barrier – The Commissioner of Railroads has determined that the Transportation Agency has authority to determine safety barriers according to their standards. The railroad overpass parapets should be designed the same as highway grade separation structures using solid parapets (Type "SS" or appropriate) and pedestrian fencing where required.

38.4.1 Preliminary Plan Preparation

Standard for Highway over Railroad Design Requirements shows the minimum dimensions for clearances and footing depths. These should be shown on the Preliminary Plan along with the following data.

- Milepost and Direction - Show the railroad milepost and the increasing direction.

- Structure Location - Show location of structure relative to railroad right of way. (Alternative is to submit Roadway Plan).

- Footings - Show all footing depths. Minimum footing depth requirements are shown on the Standard for Highway over Railroad Design Requirements.

- Drainage Ditches - Show ditches and direction of flow.

- Utilities - Show all utilities that are near structure footings and proposed relocation is required.

- Crash Protection – See Standard for Highway over Railroad Design Requirements for crash protection requirements. On a structure widening a crashwall shall be added if the multi-columned pier is equal to or less than 25 feet from centerline of track.
- Shoring – If shoring is required, use a General Note to indicate the location and limit.
- Limits of Railroad Right-of-Way – The locations are for reference only and need not be dimensioned.

38.4.2 Final Plans

The Final Plans must show all the approved Preliminary Plan data and be signed and/or sealed by a Registered Engineer.

38.4.3 Shoring

Railroad companies are particularly concerned about their track elevations. It is therefore very important that shoring is used where required and that it maintains track integrity.

38.4.4 Horizontal and Vertical Clearances

38.4.4.1 Horizontal Clearance

The distance from the centerline of track to the face of back slopes at the top of rail must not be greater than 20’-0” since federal funds are not eligible to participate in costs for providing greater distances unless required by site conditions. Minimum clearances to substructure units are determined based on site conditions and the character of the railroad line. Consideration must be given to the need for future tracks. Site specific track drainage requirements and possible need for an off-track roadway must also be considered.

38.4.4.2 Vertical Clearance

Section 192.31, Wisconsin Statutes requires 23’-0” vertical clearance above top of rail (ATR) for new construction or reconstruction, unless the Office of the Commissioner of Railroads approves less clearance. As a result, early coordination with the Railroads and Harbors Section is required.

Double stack containers at 20’-2” ATR are the highest equipment moving in restricted interchange on rail lines which have granted specific approval for their use. Allowing for tolerance, this equipment would not require more than 21’-0” ATR clearance. Railroad companies desire greater clearance for maintenance purposes and to provide allowance for possible future increases in equipment height.

38.4.4.3 Compensation for Curvature

Where a horizontal clearance obstruction is within 80 feet of curved track AREMA specifications call for lateral clearance increases as stated in *AREMA Manual* Chapter 28, Table 28-1-1.

38.4.4.4 Constructability

The minimum clearances discussed are to finished permanent work. Most railroad companies desire minimum temporary construction clearances to forms, falsework or track protection of 12'- 0" horizontal and 21'-0" vertical. The horizontal clearance provides room for a worker to walk along the side of a train and more than ample room for a train worker who may be required to ride on the side of a 10'-8" wide railroad car. Where piers are to be located close to tracks the type of footing to be used must be given careful consideration for constructability. The depth of falsework and forms for slab decks may also be limited by temporary vertical clearance requirements.

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39.1 General

WisDOT policy item:

The design for sign structures shall be in accordance with the *AASHTO Standard Specifications for Highway Bridges, 17th Edition*.

Signing is an integral part of the highway plan and as such is developed with the roadway and bridge design. Aesthetic as well as functional considerations are essential to sign structure design. Supporting sign structures should exhibit clean, light, simple lines which do not distract the motorist or obstruct his view of the highway. In special situations sign panels may be supported on existing or proposed grade separation structures in lieu of an overhead sign structure. Aesthetically this is not objectionable if the sign does not extend below the girders or above the top of the parapet railing. Some of the more common sign support structures are shown in the following figure.

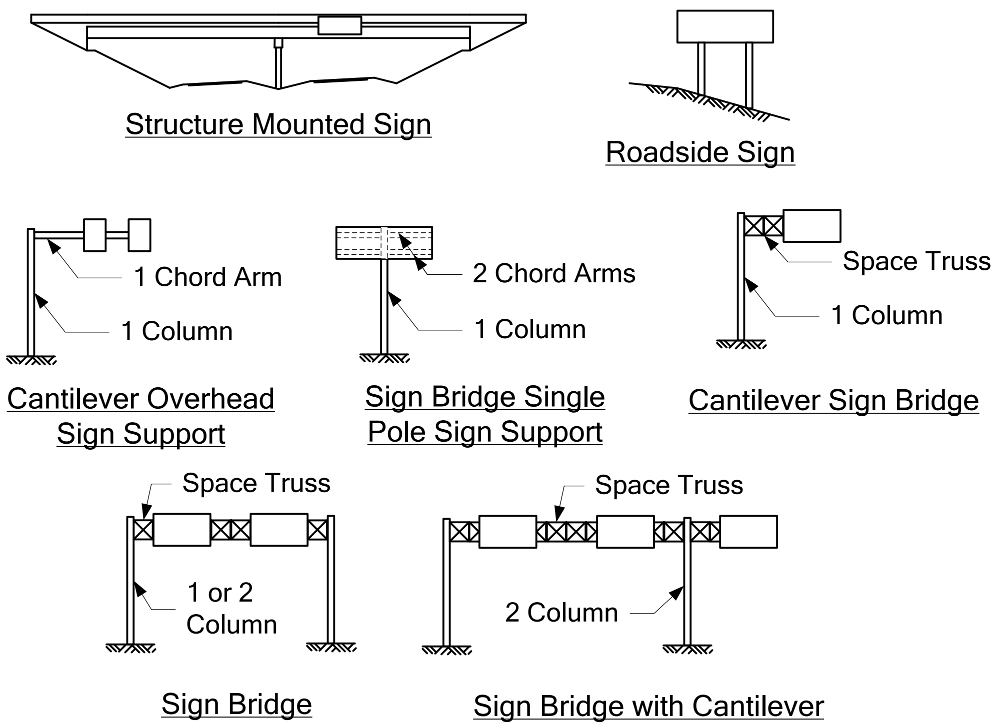

Figure 39.1-1
Sign Support Structures

39.1.1 Signs on Roadway

Roadside sign supports are located behind existing or planned guardrail as far as practical from the roadway out of the likely path of an errant vehicle. If roadside signs are located within the 30 foot corridor and not protected, break-away sign supports are detailed. Wisconsin has experienced that the upper hinge on ground mounted signs with break-away supports does not work and it is not used. Since FHWA has not approved this removal, the hinge is used on all federal projects. DMS, which includes both dynamic message signs and variable message signs, roadside sign type supports are to be protected by concrete barrier or guardrail. All overhead sign-column type supports are located at the edge of shoulder adjacent to the traveled roadway or placed behind barrier type guardrail. See the Facilities Development Manual (FDM) 11-55-20.5 for details on shielding requirements. When protection is impractical or not desirable, the uprights shall be designed with applicable extreme event collision loads in accordance to Section 13.4.10 of this manual.

Overhead sign structures, for new and replacement structures only, are to have a minimum vertical clearance of 18’-3” above the roadway. See FDM, Procedure 11-35-1 Attachment 1.9, for clearances relating to existing sign structures. The minimum vertical clearance is set slightly above the clearance line of the overpass structure for signs attached to existing structures.

39.1.2 Signs Mounted on Structures

Signs are typically installed along the major axes of a structure. Wisconsin has allowed sign attachment up to a maximum of a 20 degree skew. Any structure with greater skew requires mounting brackets to attach signs perpendicular to the roadway.

39.1.2.1 Signs Mounted on the Side of Structures

In addition to aesthetic reasons, signs attached to the side of bridge superstructures and retaining walls are difficult to inspect and maintain due to lack of access. Attachment accessories are susceptible to deterioration from de-icing chemicals, debris collection and moisture; therefore, the following guidance should be considered when detailing structure side mounted signs and related connections:

1. Limit the sign depth to a dimension equal to the bridge superstructure depth (including parapet) minus 3 inches.

2. Provide at least two point connections per supporting bracket.

3. Utilize cast-in-place anchor assemblies to attach sign supports onto new bridges and retaining walls.

4. Galvanized or stainless steel adhesive concrete masonry anchor type L may be used to attach new signs to the vertical face of an existing bridge or retaining wall. Overhead installation is not allowed. Reference Section 40.16 for applicable concrete masonry anchor requirements.

39.1.2.2 Overhead Structure Mounted Signs

Span deflections of the superstructure due to vehicle traffic are felt in overhead sign structures mounted on those bridges. The amount and duration of sign structure deflections is dependent on the stiffness of the girder and deck superstructure, the location of the sign on the bridge, and the ability of the sign structure to dampen those vibrations out; among others. These vibrations are not easily accounted for in design and are quite variable in nature. For these reasons, the practice of locating overhead sign structures onto bridges should be avoided whenever possible.

The following general guidance is given for those instances where locating a sign structure onto a bridge structure is unavoidable, which may be due to the length of the bridge, or a safety need to guide the traveling public to upcoming ramp exits or into specific lanes on the bridge.

1. Locate the sign structure support bases at pier locations.

2. Build the sign structure base off the top of the pier cap.

3. Provide set back of the upright support of the sign structure behind the back face of the parapet to preclude snagging of any vehicle making contact with the parapet.

4. Use single pole sign supports (equal balanced butterfly's) in lieu of cantilevered (with an arm on only one side of the vertical support) sign supports.

5. Consider the use of a Stockbridge type damper in the horizontal truss of these structures.

6. Do not straddle the pier leaving one support on the pier and one support off the pier in the case of skewed substructure units for full span sign bridges.

39.2 Specifications and Standards

Reference specifications for sign structures are as follows:

- AASHTO "*Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, 6th Edition*"
- AASHTO "*Standard Specifications for Highway Bridges, 17th Edition*"
- State of Wisconsin "*Standard Specifications for Highway and Structure Construction*"
- ASTM "*Standards of the American Society for Testing and Materials*"

Standard details for full span 4-chord galvanized steel sign bridge, design data and details for galvanized cantilever steel sign truss and footing are given on the Chapter 39 Standard Details.

Standard details for overhead sign support bases are provided in the Standard Detail Drawing (SDD) sheets of the FDM.

Standard design data and details for break-away sign supports and sign attachment are given on the A Series of the Sign Plate Manual.

39.3 Materials

Wisconsin has historically specified API Spec. 5L, grade 42 pipe as the primary material for the design of sign bridge chords and columns. However, due to recent supply shortage, ASTM A500 grades B and C, and ASTM A53 grade B types E and S round HSS or pipe (tubular shapes) are allowed as alternate materials for sign bridge truss main members (chords and columns) less than 10 inches diameter. API Spec. 5L, grade 42 remains the preferred material for single column uprights on both full span and cantilever sign bridges due to the toughness requirement to address fatigue concerns and the non-redundant nature of these structures. All plates, bars and structural angles shall be ASTM A709 grade 36. ASTM A595 grade A, A572, and A1011 have been used by manufacturers to design round, tapered steel tube members for overhead sign support arms and uprights. When tubular shapes are used for overhead sign supports, they shall conform to the sign bridge requirements. Unless noted otherwise in the contract plans, all bolted connections for sign structures shall be made with direct tension indicating (DTI) washers and meet the applicable requirements of high strength A325 bolts as stated in Section 24.2 of this manual. More details can be found in the Standard Drawings and Standard Specifications Section 641.

39.4 Design Considerations

39.4.1 Signs on Roadway

Supports for roadside signs are of three types, depending upon the size and type of the sign to be supported. For small signs, the column supports are treated timber embedded in the ground. For larger type I signs and DMS, the columns shall be galvanized steel supported on cylindrical concrete footings. Currently, all steel column supports for roadside type I signs are designed to break-away upon impact, while DMS supports are protected and designed without a break-away system.

The following design data is employed for designing ground mount or roadside sign supports:

Wind Velocity	75 mph based on the fastest mile wind speed map and its corresponding methods to find wind pressure.
Wind Components	Normal = 1.0
	Transverse = 0.0
Ice Load	3 psf
Group Loads	% of Allowable Stress
I Dead	100
II Dead + Wind	140
III Dead + Ice + (1/2 Wind)*	140
Allowable Soil Pressure	3 ksf

* Minimum Wind Load = 25 psf

Wind loading is applied to the area of sign and supporting members.

Ice loading is applied to one face of the sign and around the surface of supporting members.

39.4.2 Overhead Sign Structures

Sign structures for support of overhead signs consist of “sign bridges” and “overhead sign supports”. Sign bridges are to be either a single column cantilever or butterfly, or a space truss sign bridge supported by one or two columns at each end. For cantilever sign bridge structures, the footing is a single cylindrical shaft with wings to prevent the overturning and twisting of the structure. For space trusses having one or two steel columns on an end, the footing is composed of two cylindrical caissons connected by a concrete cross-girder. The top surface of concrete foundations for all sign bridges is to be located 3' above the highest ground line at the foundation. Occasionally, some sign bridge columns are mounted directly on top of modified bridge parapets, pier caps and concrete towers instead of footings.

Sign bridges also include sign support members mounted directly onto structures. Sign attachments, such as galvanized steel I-beams and/or brackets, typically are anchored to the side of the bridge superstructure. A cantilever truss attached to the side of retaining walls (without a vertical column) is also common.

Similar to sign bridges, all overhead sign supports have single galvanized steel column supported on a cylindrical caisson footing or on top of bridge elements. Cross members can be one chord (monotube), two chords without web elements, or planar truss in either cantilever or full span structure.

The following design data is employed for designing steel sign bridges and overhead sign supports.

Wind Velocity = 90 mph based on the 3-second gust wind speed map and its corresponding methods to find wind pressure.

Wind Components	Normal	Transverse
Combination 1	1.0	0.2
Combination 2	0.6	0.3

Table 39.4-1
Wind Components

Dead Load = Wt. of Sign, supporting structure, catwalk and lights.

Ice Load = 3 psf to one face of sign and around surface of members.

Group Loads	% of Allowable Stress
I Dead	100
II Dead + Wind	133
III Dead + Ice + (1/2 Wind)*	133
IV Fatigue	**

Table 39.4-2
Allowable Overstresses

* Minimum Wind Load = 25 psf

** See Fatigue section of AASHTO for fatigue loads and stress range limits.

WisDOT policy item:

Fatigue group loads application is exempt on 4-chord full span sign bridges with truss type uprights mounted on concrete footings.

Steel cantilevered sign bridge structures (4-chord structures carrying type I signage) are classified, for purposes of fatigue design, as Category 1 structures. These cantilevered support structures are designed to resist Natural Wind Gust and Truck-Induced Gust wind effects. 4-

chord cantilevered sign supports carrying type I signage are not designed for Galloping wind effects due to the substantial stiffness and satisfactory performance history in this state.

Steel cantilever sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. Columns are made from pipe sections. The minimum thickness for the members is indicated on the steel cantilever Standard detail.

Steel full span sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. The minimum thickness of steel web members is 3/16 inch and 0.216 inches for chord members. The connections of web members to chords are designed for bolting or shop welding to allow the contractor the option to either galvanize individual members or complete truss sections after fabrication. The upright columns are either steel pipe or tubular shape sections with web members (planar truss), see Section 39.3 for additional details. Steel base plates are used for anchor bolt support attachment.

When butt welding box sections, a back-up plate is required since the plates can only be welded from one side. The plate must be of adequate width for film to be used during weld inspection. The exposed weld is ground smooth for appearance as well as fatigue.

Aluminum sign bridges are currently not being designed for new structures. Rehabilitation and repair type work may require use of aluminum members and shall be allowed in these limited instances. The following guidelines apply to aluminum structures in the event of repair and rehabilitation type work.

Aluminum sign bridge trusses are designed and fabricated from tubular shapes shop welded together in sections. The minimum thickness of truss chords is ¼ inch and the minimum outside diameter is 4 inches. The recommended minimum ratios of “d/D” between the outside diameters “d” of the web members and “D” of chord members is 0.4. A cast aluminum base plate is required to connect the aluminum columns to the anchor bolts. AASHTO Specifications require damping or energy absorbing devices on aluminum overhead sign support structures to prevent vibrations from causing fatigue failures. Damping devices are required before and after the sign panels are erected on all aluminum sign bridges. Stock-bridge type dampers are recommended.

Install permanent signs to sign structures at the time of erection. If the signs are not available, install sign blanks to control vibration. For sign bridges, blanks are attached to a minimum of one-fourth the truss length near its center. The minimum depth of the blanks is equal to the truss depth plus 24 inches. The blanks are to be installed to project an equal dimension beyond the top and bottom chord members. Overhead sign support blanks are equal to the same sizes and at the same locations as the permanent signs. Contact BOS Structures Design Section at 608-267-2869 for further guidance on other vibration controlling methods.

Do not add catwalks to new sign bridges unless they contain DMS over traffic. Catwalks add additional cost to a structure and present a maintenance issue. They can be added if a decision is made to light the signs in the future. Design structures with type I and II signs for a 20’-3” (2’-0” additional) vertical clearance when they are located in a continuous median freeway lighting area, for new and replacement sign bridges only. Structures with DMS may require larger vertical clearance to the bottom of the sign depending on the type of catwalk being designed for future installation. The sign bridge should be structurally designed to support a catwalk for those cases when the additional clearance is provided for possible future attachment. Additional accommodations for potential future lighting include providing hand holes in the uprights, rodent screens and conduits in the concrete bases.

For structures that are not located in continuous median freeway lighting areas or do not contain DMS, the additional structure height should not be utilized. Therefore, the design vertical clearance should be 18’-3” for new and replacement sign bridges only. No hand holes, rodent screens or conduits shall be installed on the structure in this case. However, all DMS sign bridges require hand holes, rodent screens and conduits.

Brackets, if required, for maintenance of light units are required to support a 2'-3” wide catwalk grating and a collapsible aluminum handrail. Brackets and handrailing for type I and II signs are fabricated from aluminum sections, whereas DMS support brackets are made of galvanized steel. Catwalk grating and toe plates are fabricated from steel and shall be galvanized.

Contract plans should note (under the general notes) if hand holes are required on one or both uprights of the sign bridge.

Overhead sign supports are typically not lit, nor do they require sign maintenance. Therefore, do not detail a catwalk on this type of structure. Also do not detail hand holes, rodent screens or conduits unless the structure is designed to carry an LED changeable message sign.

Design of all Sign Bridge structures should reflect some provision for the possibility of adding signs in the future (additional sign area). Consideration should include the number of lanes, possible widening of roadway into the median or shoulder areas, and use of diagrammatic signs to name a few. The truss design should reflect sizing the chords for maximum force at the center of the span. The design of the upright and truss webs should allow for signs being placed (say sometime in the future) more skewed to one side than the other. Uprights should be selected the same size (outside diameter x thickness) for each side and the design shall reflect different lengths on either side as required by site conditions.

The design sign area and maximum sign depth dimensions for type I and II signs shall be explicitly listed with the design data in the contract plans. Use 3 psf dead load for these types of signs. Provide manufacturer overall DMS dimensions in the plans along with the total weight of the signs. Other loads such as Catwalks, lights and associated attachments must also be included in the overall design data in the contract plans.

The following guidance is recommended for estimating design sign areas.

1. Type I and II signs on full span sign bridges, design sign area equals the largest value resulting from the four requirements below:

    a. Total actual sign area.

b. Two (2) times the controlling upright tributary sign area. Tributary area is computed based on the application of the lever rule on a simply supported truss.

c. Twelve (12) times the number of lanes times the maximum sign depth. The number of lanes is defined as the clear roadway width (including median and shoulders) divided by 12 and rounding down to the nearest whole number.

d. Maximum sign depth times 60% of the span length (center to center of upright).

For design purposes, the standard sign depth shall be limited between 12'-0" and 16'-0". Therefore, vertical clearance and upright lengths are to be sized with sign depth not less than 12'-0", unless requested otherwise in the structure survey report. Mega projects with series of sign bridges may deviate from the above requirements provided that coordination is made with the BOS Structures Design Section.

2. Type I and II maximum design sign area for galvanized steel cantilever sign truss is detailed in Standard 39.10. Sizing the upright length and vertical clearance with 12'-0" sign depth is recommended for future accommodation.

3. DMS sign bridges should be designed with the actual sign dimensions in addition to those of type I and II signs and catwalk as applied.

4. Overhead sign supports are generally designed with the actual sign dimensions and locations. Exception to the approach may be granted to structures with anticipated change in signage.

39.5 Structure Selection Guidelines

Sign structures are composed of “sign bridges” and “overhead sign supports”. Either type of sign structure can be configured to be a cantilever sign structure (one upright to arm) or a full-span sign structure (two uprights, one on each end of the span). Roadside sign supports are an exception to the above naming convention.

“Sign Bridges” generally carry type I and II signs, and occasionally DMS. These are large sign structures with sign depths ranging from 5’-0” or less to 18’-0” in the case of large diagrammatic signs. Total sign areas accommodated are up to 264 sq. ft. on cantilever sign bridges. Total sign areas accommodated on full span sign bridges range from 250 to over 1000 sq. ft. of sign area. These ranges are an approximate guide only. Most sign bridges generally have truss members consisting of four round chords and angle web members supporting type I and II signs on the span or arm (although some three chord structures have been used for full span sign bridges). Uprights are comprised of one column for a cantilever sign bridge. Full span sign bridge uprights usually consist of two columns joined by angle web members at each end of the span (although single column uprights have been used on three chord full span sign bridges). “Sign Bridges” are designed by the Bureau of Structures or a consultant. Structure contract plans provide full details that a fabricator can construct the sign bridge from. Standard details for the 4-chord sign bridge associated with this Chapter of the Bridge Manual require a design for each sign bridge structure including foundations. These details are used for type I and II sign applications only.

Sign bridges carrying variable message signs require special consideration. Special concerns include:

1. Weight of the sign panel.

2. Width and weight of catwalk.

3. Consideration of wind effects unique to these signs.

4. Modification to brackets used. All catwalk and sign connection brackets shall be made with friction type connections and high strength A325 bolts with DTI washers.

Wisconsin recommends the use of the Minnesota 4-chord configuration for sign bridges carrying DMS, providing that the designer checks the design of each member and connection details conform to the latest AASHTO Standard specification requirements.

“Overhead Sign Supports” are smaller sign structures carrying both type II (smaller) directional signs, limited amounts of type I signs and small LED or changeable message signs. Type II sign depths have ranged from 3’-0” to 4’-0” deep for traffic directional signs, and up to 10’-0” for small information type I signs. When a sign is larger than 10’-0” deep, the structure is to be designed as a sign bridge. Cantilever overhead sign supports accommodate up to 45 sq. ft. of sign area. Total sign areas accommodated on full span overhead sign supports range up to 300 sq. ft. These ranges are again an approximate guide and can be more or less depending on variables such as span length, location of the sign with respect to the upright(s), the height of the upright(s), etc. Uprights are comprised of one pole (uniform round or tapered pipe) for either the cantilever or full span overhead sign support. Arms on cantilevers or the span on a full span overhead sign support are either one chord (uniform round or tapered pipe), or two chords with or without angle web members depending on the span length and sign depth. Due to the variability of factors that can influence the selection of structure type, designers are encouraged to contact BOS Structures Design Section for further assistance when sign areas fall outside of the above limits, or structural geometry is in question. "Overhead Sign Supports" are normally bid by the contractor and designed by a fabricator or by another party for a fabricator to construct. Typical structures with steel poles on standard concrete bases usually have the least plan detail associated with them and are normally depicted in the Construction Detail portion of the state contract plans. However, it is recommended that plan development for projects with multiple structures, such as major or mega projects, and structures mounted on non-standard supports to be prepared by structural engineers and placed in section 8 of the contract plans along with the sign bridge plans. When a standard concrete footing base design is required the drawing must be shown in the contract plans for overhead sign supports. See the WisDOT FDM Procedures 11-55-20 and 15-1-20 for more information on Overhead Sign Supports.

39.6 Geotechnical Guidelines

Several potential problems concerning the required subsurface exploration for foundations of sign structures exist. These include:

- The development and location of these structures are not typically known during the preliminary design stage, when the majority of subsurface exploration occurs. This creates the potential for multiple drilling mobilizations to the project.

- Sometimes these structures are located in areas of proposed fill soils. The source and characteristics of this fill soil is unknown at the time of design.

- The unknowns associated with these structures in the scoping/early design stages complicate the consultant contracting process. How much investigation should be scoped in the consultant design contract?

Currently, all sign structure foundations are completely designed and detailed in the project plans. Sign-related design information can be found in the Facilities Development Manual (FDM) or Bridge Manual as described in the following sections.

39.6.1 Sign Bridges

WisDOT has created a standard foundation design for cantilever sign bridges carrying Type I signs. This standard foundation is presented on Standard 39.12 of the Bridge Manual Standard Details. The wings on this single shaft footing are used to help resist torsion. If the cantilever sign bridge, carrying Type I signs, exceeds the criteria/limitations (shown on Standard 39.10), the standard foundation shall not be utilized, and an individual foundation must be fully designed. This customized design will involve~~s~~ determining the subsurface conditions as described in section 39.6.3.

Foundations supporting all full span sign bridges are custom designed. They generally have two cylindrical shafts connected by a concrete cross-girder below each vertical upright. WisDOT has no standard details for the foundations for these structures.

WisDOT policy item:

The length of a cast-in-place shaft foundation shall be limited to 20’-0”. Deviation from this policy item may be allowed provided coordination is made with BOS Structures Design Section.

39.6.2 Overhead Sign Supports

Overhead sign supports are described in Sections 11-55-20 and 15-1-20 of the FDM. In addition, Section 641 of the Standard Specifications outlines the design/construction aspects of these structures.

If these structures meet several criteria/limitations that are listed on the SDD’s, the designer can use WisDOT-developed standard foundations for them. The designer can then insert the proper SDD sheet into the plans. SDD sheets exist for cantilever overhead sign supports.

These single shaft bases for cantilever overhead sign supports vary in depth and range from 24” to 42” in diameter. Another SDD sheet applies to full-span overhead sign supports and is 36” in diameter. The standard foundations in these SDD sheets were designed using slightly conservative soil design parameters. If the design criteria for these standard designs are not met, the SDD sheets cannot be used and the structure foundation must be fully designed and the unique details supplied in the construction details portion of the contract plans. This involves determining the subsurface conditions as described in the following section.

39.6.3 Subsurface Investigation and Information

No subsurface investigation/information is necessary for any of the sign structures that meet the limitations for allowing the use of WisDOT standard foundations. Appropriate subsurface information is necessary for any of these structures that require custom designs.

There may be several methods to obtain the necessary subsurface soil properties to allow for a custom design of foundations, as described below:

- In areas of fill soils, the borrow material may be unknown. The designer should use their best judgment as to what the imported soils will be. Standard compaction of this material can be assumed. Conservative soil design parameters are encouraged.

- The designer may have a thorough knowledge of the general soil conditions and properties at the site and can reasonably estimate soil design parameters.

- The designer may be able to use information from nearby borings. Judgment is needed to determine if the conditions present in an adjacent boring(s) are representative of those of the site in question.

- If the designer cannot reasonably characterize the subsurface conditions by the above methods, a soil boring and Geotechnical report (Site Investigation Report) should be completed. Necessary soil design information includes soil unit weights, cohesions, phi-angles and location of water table.

Designers, both internal and consultant, should also be aware of the potential of high bedrock, rock fills and the possible conflict with utilities and utility trenches.

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N_L = number of design lanes as specified in **LRFD [3.6.1.1.1]**

For single-lane loading:

(Span 1, 3) $E := 10.0 + 5.0\cdot(38\cdot 30)^{0.5}$ $\boxed{E = 178.819}$ in

(Span 2) $E := 10.0 + 5.0\cdot(51\cdot 30)^{0.5}$ $\boxed{E = 205.576}$ in

For multi-lane loading:

$$12.0\cdot\frac{W}{N_L} = 12.0\cdot\frac{42.5}{3} = 170 \quad \text{in}$$

(Span 1, 3) $E := 84.0 + 1.44\cdot(38\cdot 42.5)^{0.5}$ $\boxed{E = 141.869}$ in <170" O.K.

(Span 2) $E := 84.0 + 1.44\cdot(51\cdot 42.5)^{0.5}$ $\boxed{E = 151.041}$ in <170" O.K.

E45-1.2.3 Nominal Flexural Resistance: (M_n)

The depth of the compressive stress block, (a) is (See 18.3.3.2.1):

$$a = \frac{A_s\cdot f_s}{\alpha_1\cdot f'_c\cdot b}$$

where:

A_s = area of developed reinforcement at section (in^2)

f_s = stress in reinforcement (ksi)

$f'_c = 4$ ksi

$b := 12$ in

$\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi) **LRFD [5.7.2.2]**

As shown throughout the Chapter 18 Example, when f_s is assumed to be equal to f_y, and is used to calculate (a), the value of c/d_s will be < 0.6 (for f_y = 60 ksi) per **LRFD [5.7.2.1]** Therefore the assumption that the reinforcement will yield ($f_s = f_y$) is correct. The value for (c) and (d_s) are calculated as:

$$c = \frac{a}{\beta_1}$$

$\beta_1 := 0.85$

d_s = slab depth(excl. 1/2" wearing surface) - bar clearance - 1/2 bar diameter

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals:

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

Minimum Reinforcement Check

All sections throughout the bridge meet minimum reinforcement requirements, because this wa checked in the chapter 18 Design example. Therefore, no adjustment to nominal resistance (M_n) or moment capacity is required. **LRFD [6.5.7]**

Maximum Reinforcement Check

The area of reinforcement to be used in calculating nominal resistance (M_n) or moment capacity, shall not exceed the maximum amount permitted in **LRFD [5.7.3.3.7]**, as stated in **LRFR[6.5.6]**. This check will be ignored because the article referenced in the *LRFD Specifications*, as mentioned above, has been removed.

E45-1.2.4 General Load - Rating Equation (for flexure)

$$RF = \frac{C - (\gamma_{DC}) \cdot (M_{DC}) - (\gamma_{DW}) \cdot (M_{DW})}{\gamma_L \cdot (M_{LL\_IM})}$$ **LRFR [6.4.2.1]**

For the Strength Limit State:

$$C = (\phi_c)(\phi_s)(\phi) \cdot R_n$$

where:

$R_n = M_n$ (for flexure)

$(\phi_c)(\phi_s) \geq 0.85$

Factors affecting Capacity (C):

Resistance Factor (ϕ), for Strength Limit State **LRFR [6.5.3]**

$\phi := 0.9$ for flexure (all reinforced concrete section in the Chapter 18 Example were found to be tension-controlled sections as defined in **LRFD [5.7.2.1]**).

Condition Factor (ϕ_c) per Chapter 45.3.2.4

$\phi_c := 1.0$

System Factor (ϕ_s) Per Chapter 45.3.2.5

$\phi_s := 1.0$ for a slab bridge

E45-1.2.5 Design Load (HL-93) Rating

Use Strength I Limit State to find the Inventory and Operating Ratings **LRFR [6.4.2.2, 6.5.4.1]**

Equivalent Strip Width (E) and Distribution Factor (DF):

Use the smaller equivalent width (single or multi-lane), when (HL-93) live load is to be distributed, for Strength I Limit State. Multi-lane loading values will control for this bridge.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E} \qquad \text{(where E is in feet)}$$

The multiple presence factor, m, has been included in the equations for distribution width, E, and therefore is not used to adjust the distribution factor, DF, **LRFD [3.6.1.1.2]**.

Spans 1 & 3:

DF = 1/(141"/12) = 0.0851 lanes / ft-slab

Span 2:

DF = 1/(151"/12) = 0.0795 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.
Therefore use: $DF := 0.0851$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM := 33$ % **LRFR [6.4.3.3]**

Live Loads (LL)

The live load combinations used for Strength I Limit State are shown in the Chapter 18 Example in Table E18.2 and E18.3. The unfactored moments due to Design Lane, Design Tandem, Design Truck and 90%{Double Design Truck + Design Lanes] are shown in Chapter 18 Example (Table E18.4).

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_{Li} := 1.75$ (Inventory Rating) Chapter 45 Table 45.3-1

$\gamma_{Lo} := 1.35$ (Operating Rating) Chapter 45 Table 45.3-1

The Design Load Rating was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location, for this example, is in span 1 at the 0.4 pt.

Span 1 (0.4 pt.)

Inventory:

$$RF_i = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$A_{st\_0.4L} = 1.71 \ \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 17.0 - 1.5 - 0.6$ $\boxed{d_s = 14.9}$ in

$a := \frac{A_{st\_0.4L}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.51}$ in

$M_n := A_{st\_0.4L}\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 1399.7}$ kip – in

$\boxed{M_n = 116.6}$ kip – ft

$M_{DC} := 18.1$ kip – ft (from Chapter 18 Example, Table E18.4)

$M_{DW} := 0.0$ kip – ft (additional wearing surface not for HL-93 rating runs)

The positive live load moment shall be the largest caused by the following (from Chapter 18 Example, Table E18.4):

Design Tandem (+IM) + Design Lane: (37.5 kip-ft + 7.9 kip-ft) = 45.4 kip-ft
Design Truck (+IM) + Design Lane: (35.4 kip-ft + 7.9 kip-ft) = 43.3 kip-ft

Therefore:

$M_{LL\_IM} := 45.4$ kip – ft

Inventory:

$$RF_i := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Li}\cdot(M_{LL\_IM})}$$

$\boxed{RF_i = 1.04}$

Operating:

$$RF_o := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_{Lo}\cdot(M_{LL\_IM})}$$

$\boxed{RF_o = 1.34}$

Rating for Shear:

Slab bridge designed for dead load and (HL-93) live load moments in conformance with **LRFD [4.6.2.3]** may be considered satisfactory in shear **LRFD [5.14.4.1]**. This bridge was designed using this procedure, therefore a shear rating is not required.

The Rating Factors, RF, for Inventory and Operating Rating are shown on the plans and also on the load rating summary sheet.

E45-1.2.6 Permit Vehicle Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.6).

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic load allowance is utilized. Future wearing surface will not be considered.

Since this example is rating a newly designed bridge, and additional check is required. The designer shall ensure that the results of a single-lane analysis utilizing the future wearing surface are greater than 190 kips MVW.

Use Strength II Limit State to find the Permit Vehicle Load Rating **LRFR [6.4.2.2, 6.5.4.2.1]**.

E45-1.2.6.1 Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **LRFR [6.4.5.4.2.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **LRFR [6.3.2, C6.4.5.4.2.2, Table 6-6]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E\cdot(1.20)}$$ (where E is in feet)

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value.
This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33$ % **LRFR [6.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_{DW} := 1.50$ WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88\ \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_C \le 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 28.0 - 2.0 - 0.5$ $\boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ in

$M_n := A_{st\_pier}\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$ $\boxed{M_n = 2720.5}$ kip – in

$\boxed{M_n = 226.7}$ kip – ft

$M_{DC} := 59.2\ kip - ft$ (from Chapter 18 Example, Table E18.4)

$M_{DW} := 1.5 \text{ kip} - \text{ft}$

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$M_{LL\_IM} := 65.2 \quad \text{kip} - \text{ft}$

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$$\boxed{RF_{permit} = 1.63}$$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit}\cdot(190) = 310 \quad \text{kips}$ which is > 190k, Check OK

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

Rating for Shear:

WisDOT does not rate Permit Vehicles on slab bridges based on shear.

E45-1.2.6.2 Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

Equivalent Strip Width (E) and Distribution Factor (DF)

The equivalent width from single-lane loading is used, when Permit Vehicle live load is to be distributed, for Strength II Limit State **LRFR [6.4.5.4.2.2]**.

Calculate the distribution factor, DF, and divide it by (1.20) to remove the effects of the multiple presence factor (m), which are present in the equation for equivalent width (E) **LRFR [6.3.2, C6.4.5.4.2.2, Table 6-6]**.

The distribution factor, DF, is computed for a slab width equal to one foot.

$$DF = \frac{1}{E\cdot(1.20)} \qquad \text{(where E is in feet)}$$

Spans 1 &3:

DF = 1/(178"/12)(1.20) = 0.0562 lanes / ft-slab

Span 2:

DF = 1/(205"/12)(1.20) = 0.0488 lanes / ft-slab

Look at the distribution factor calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Therefore use: $DF := 0.0562$ lanes / ft-slab for all spans.

Dynamic Load Allowance (IM)

$IM = 33$ % **LRFR [6.4.5.5]**

Rating for Flexure

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

Load Factors

$\gamma_{DC} := 1.25$ Chapter 45 Table 45.3-1

$\gamma_L := 1.20$ WisDOT Policy is to designate the (Wis_SPV) as a "Single-Trip" vehicle with no escorts. Current policy is to select the value for γ_L from Chapter 45 Table 45.3-3

The Maximum Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is the C/L of Pier.

At C/L of Pier

Permit Vehicle:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88 \frac{in^2}{ft}$ and $\alpha_1 := 0.85$ (for $f'_c \leq 10.0$ ksi) **LRFD [5.7.2.2]**

$d_s := 28.0 - 2.0 - 0.5$ $\boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{\alpha_1\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ in

* Dead load on composite (DC_2):

weight of single parapet, klf $\boxed{w_p = 0.387}$ klf

weight of 2 parapets, divided equally to all girders, klf

$$DC_2 := \frac{w_p \cdot 2}{ng}$$ $\boxed{DC_2 = 0.129}$ klf

$$V_{DC2} := \frac{DC_2 \cdot L}{2}$$ $\boxed{V_{DC2} = 9}$ kips

$$M_{DC2} := \frac{DC_2 \cdot L^2}{8}$$ $\boxed{M_{DC2} = 344}$ kip-ft

* Wearing Surface (DW): There is no current wearing surface on this bridge. However, it is designed for a 20 psf future wearing surface. Thus, it will be used in the calculations for the Wisconsin Standard Permit Vehicle Design Check, Section 45.6.

$$DW := \frac{w \cdot 0.020}{ng}$$ $\boxed{DW = 0.133}$ klf

$$V_{DW} := \frac{DW \cdot L}{2}$$ $\boxed{V_{DW} = 10}$ kips

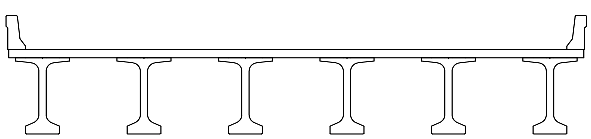
$$M_{DW} := \frac{DW \cdot L^2}{8}$$ $\boxed{M_{DW} = 355}$ kip-ft

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

E45-2.5 Live Load Analysis - Interior Girder

Live Load Distribution Factors (g)

In accordance with **LRFD [Table 4.6.2.2.1-1]**, this structure is a Type "K" bridge.



Distribution factors are in accordance with **LRFD [Table 4.6.2.2.2b-1]**. For an interior beam, the distribution factors are shown below:

For one Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

For Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1}$$

E45-2.5.1 Moment Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{i1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i1} = 0.435}$$

Two or More Lanes Loaded:

$$g_{i2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12.0 \cdot L \cdot t_{se}^{\ 3}}\right)^{0.1} \qquad \boxed{g_{i2} = 0.636}$$

$$g_i := \max(g_{i1}, g_{i2}) \qquad \boxed{g_i = 0.636}$$

Note: The distribution factors above already have a multiple presence factor included that is used for service and strength limit states. For permit load analysis utilizing single lane distribution, the 1.2 multiple presence factor should be divided out.

E45-2.5.2 Shear Distribution Factors for Interior Beams:

One Lane Loaded:

$$g_{v1} := 0.36 + \frac{S}{25} \qquad \boxed{g_{v1} = 0.660}$$

Two or More Lanes Loaded:

$$g_{v2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \boxed{g_{v2} = 0.779}$$

$$g_v := \max(g_{v1}, g_{v2}) \qquad \boxed{g_v = 0.779}$$

E45-2.5.3 Live Load Moments

The unfactored live load load moments (per lane including impact) are listed below (values are in kip-ft). Note that the dynamic load allowance is applied only to the truck portion of the HL-93 loads.

Unfactored Live Load + Impact Moments per Lane (kip-ft)		
Tenth Point	Truck	Tandem
0	0	0
0.1	1783	1474
0.2	2710	2618
0.3	4100	3431
0.4	4665	3914
0.5	4828	4066

The unfactored live load moments per lane are calculated by applying the appropriate distribution factor to the controlling moment. For the interior girder:

$g_i = 0.636$

$M_{LLIM} := g_i \cdot 4828$ $\boxed{M_{LLIM} = 3073}$ kip-ft

E45-2.6 Compute Nominal Flexural Resistance at Midspan

At failure, we can assume that the tendon stress is:

$$f_{ps} = f_{pu}\left(1 - k \cdot \frac{c}{d_p}\right)$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

From **LRFD Table [C5.7.3.1.1-1]**, for low relaxation strands, $k := 0.28$.

"c" is defined as the distance between the neutral axis and the compression face (inches).

Assumed dimensions:

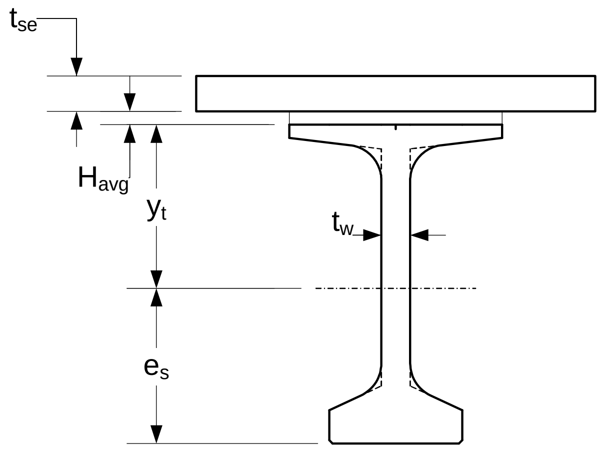


Figure E45-2.4

Assume that the compression block is in the deck. Calculate the capacity as if it is a rectangular section (with the compression block in the flange). The neutral axis location,calculated in accordance with **LRFD 5.7.3.1.1** for a rectangular section, is:

$$c = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$

where:

$A_{ps} := ns \cdot A_s$ $\qquad \boxed{A_{ps} = 9.98}$ in²

$b := b_{eff}$ $\qquad \boxed{b = 90.00}$ in

LRFD [5.7.2.2] $\alpha_1 := 0.85$ (for $f'_{cd} \le 10.0$ ksi)

$\beta_1 := \max\left[0.85 - \left(f'_{cd} - 4\right) \cdot 0.05, 0.65\right]$ $\qquad \boxed{\beta_1 = 0.850}$

$d_p := y_t + H_{avg} + t_{se} - e_s$ $\qquad \boxed{d_p = 77.15}$ in

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad \boxed{c = 9.99} \text{ in}$$

$a := \beta_1 \cdot c$ $\qquad \boxed{a = 8.49}$ in

The calculated value of "a" is greater than the deck thickness. Therefore, the rectangular assumption is incorrect and the compression block extends into the haunch. Calculate the neutral axis location and capacity for a flanged section:

$h_f := t_{se}$ depth of compression flange $\qquad \boxed{t_{se} = 7.500}$ in

$b_{tf} = 48.00$ width of top flange, inches

$$c := \frac{A_{ps} \cdot f_{pu} - \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b_{tf} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}}$$ $\boxed{c = 10.937}$ in

$a := \beta_1 \cdot c$ $\boxed{a = 9.30}$ in

This is above the base of the haunch (9.5 inches) and nearly to the web of the girder. Assume OK.
Now calculate the effective tendon stress at ultimate:

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right)$$ $\boxed{f_{ps} = 259.283}$ ksi

$T_u := f_{ps} \cdot A_{ps}$ $\boxed{T_u = 2588}$ kips

Calculate the nominal moment capacity of the composite section in accordance with **LRFD [5.7.3.2], [5.7.3.2.2]**:

$$M_n := \left[A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \alpha_1 \cdot f'_{cd} \cdot (b - b_{tf}) \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)\right] \cdot \frac{1}{12}$$

$\boxed{M_n = 15717}$ kip-ft

For prestressed concrete, $\phi_f := 1.00$, **LRFD [5.5.4.2.1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 15717}$ kip-ft

Check Minimum Reinforcement
The amount of reinforcement must be sufficient to develop M_r equal to the lesser of M_{cr} or 1.33 M_u per **LRFD [5.7.3.3.2]**

$\gamma_{LL} := 1.75$ $\gamma_{DC} = 1.250$ $\eta := 1.0$

$Mu := \eta \cdot \left[\gamma_{DC} \cdot (M_{DC1} + M_{DC2}) + \gamma_{LL} \cdot M_{LLIM}\right]$ $\boxed{Mu = 11832}$ kip-ft

$\boxed{1.33 \cdot Mu = 15737}$ kip-ft

Calculate M_{cr} next and compare its value with 1.33 Mu

M_{cr} is calculated as follows:

$f_r := 0.24 \cdot \sqrt{f'_c}$ **LRFD [5.4.2.6]** $\boxed{f_r = 0.679}$ ksi

$f_{cpe} := \dfrac{T}{A_g} + \dfrac{T \cdot e_s}{S_b}$ $\boxed{f_{cpe} = 4.414}$ ksi

$M_{dnc} := M_{DC1}$ $\boxed{M_{dnc} = 4820}$ kip-ft

$S_c := -S_{cgb}$ $\boxed{S_c = 24650}$ ksi

$S_{nc} := -S_b$ $\boxed{S_{nc} = 18825}$ ksi

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_2 := 1.1$ prestress variability factor

$\gamma_3 := 1.0$ for prestressed concrete members

$$M_{cr} := \gamma_3 \cdot \left[S_c \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}\right) \cdot \frac{1}{12} - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1\right) \right]$$ $\boxed{M_{cr} = 10713}$ kip-ft

$M_{cr} = 10713$ kip-ft $<$ $1.33Mu = 15737$, therefore M_{cr} controls

This satisfies the minimum reinforcement check since $M_{cr} < M_r$

Elastic Shortening Loss

at transfer (before ES loss) **LRFD [5.9.5.2]**

$T_{oi} := ns \cdot f_{tr} \cdot A_s$ $\boxed{= 46 \cdot 202.5 \cdot 0.217 = 2021}$ kips

The ES loss estimated above was: $\Delta f_{pES\_est} := 17$ ksi, or $ES_{loss} = 7.900$ %. The resulting force in the strands after ES loss:

$T_o := \left(1 - \dfrac{ES_{loss}}{100}\right) \cdot T_{oi}$ $\boxed{T_o = 1862}$ kips

However, since there are draped strands for a distance of $HD := 49$ from the end of the girder, a revised value of e_s should be calculated based on the estimated location of the critical section. Since the draped strands will raise the center of gravity of the strand group near the girder end, try a smaller value of "d_v" and recalculate "e_s" and "a".

Try $d_v := 65$ inches.

For the standard bearing pad of width, $w_{brg} := 8$ inches, the distance from the end of the girder to the critical section:

$$L_{crit} := \left(\frac{w_{brg}}{2} + d_v\right)\cdot\frac{1}{12} + 0.5 \qquad \boxed{L_{crit} = 6.25} \text{ ft}$$

Calculate the eccentricity of the strand group at the critical section.

$$slope = 10.274$$

$$y_{8t} := A + y_b$$

$$y_{8t} = 32.130$$

$ns_{sb} := 38$ number of undraped strands

$ns_d := 8$ number of draped strands

Find the center of gravity for the 38 straight strands from the bottom of the girder:

$$Y_S := \frac{12\cdot 2 + 12\cdot 4 + 12\cdot 6 + 2\cdot 8}{ns_{sb}} \qquad \boxed{Y_S = 4.211} \text{ in}$$

$$y_s := y_b + Y_S \qquad y_s = -30.659 \text{ in}$$

$$y_{8t\_crit} := y_{8t} - \frac{slope}{100}\cdot L_{crit}\cdot 12 \qquad \boxed{y_{8t\_crit} = 24.42} \text{ in}$$

$$e_{s\_crit} := \frac{ns_{sb}\cdot y_s + ns_d\cdot y_{8t\_crit}}{ns_{sb} + ns_d} \qquad \boxed{e_{s\_crit} = -21.08} \text{ in}$$

Calculation of compression stress block based on revised eccentricity:

$$d_{p\_crit} := y_t + H_{avg} + t_{se} - e_{s\_crit} \qquad \boxed{d_{p\_crit} = 67.71} \text{ in}$$

Note that the area of steel is based on the number of bonded strands.

$$A_{ps\_crit} := (ns)\cdot A_s \qquad \boxed{A_{ps\_crit} = 9.98} \text{ in}^2$$

Also, the value of f_{pu}, should be revised if the critical section is located less than the development length from the end of the beam. The development length for a prestressing strand is calculated in accordance with **LRFD [5.11.4.2]**:

$K := 1.6$ for prestressed members with a depth greater than 24 inches

$\boxed{d_b = 0.600}$ in

$$l_d := K \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_b \qquad \boxed{l_d = 144.6} \text{ in}$$

The transfer length may be taken as: $l_{tr} := 60 \cdot d_b$ $\boxed{l_{tr} = 36.00}$ in

Since $L_{crit} = 6.250$ feet is between the transfer length and the development length, the design stress in the prestressing strand is calculated as follows:

$$f_{pu\_crit} := f_{pe} + \frac{L_{crit} \cdot 12 - l_{tr}}{l_d - l_{tr}} \cdot \left(f_{ps} - f_{pe}\right) \qquad \boxed{f_{pu\_crit} = 198} \text{ ksi}$$

For rectangular section behavior:

$$c := \frac{A_{ps\_crit} \cdot f_{pu\_crit}}{\alpha_1 \cdot f'_{cd} \cdot \beta_1 \cdot b + k \cdot A_{ps\_crit} \cdot \frac{f_{pu\_crit}}{d_{p\_crit}}} \qquad \boxed{c = 7.349} \text{ in}$$

$$a_{crit} := \beta_1 \cdot c \qquad \boxed{a_{crit} = 6.247} \text{ in}$$

Calculation of shear depth based on refined calculations of e_s and a:

$$d_{v\_crit} := -e_{s\_crit} + y_t + H_{avg} + t_{se} - \frac{a_{crit}}{2} \qquad \boxed{d_{v\_crit} = 64.59} \text{ in}$$

This value matches the assumed value of d_v above. OK!

The nominal shear resistance of the section is calculated as follows, **LRFD [5.8.3.3]**:

$$V_n = \min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

$$\boxed{RF_{shear\_Inv} = 1.110}$$

Operating Level

$$RF_{shear\_Op} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC}\cdot(V_{DCnc} + V_{DCc})}{\gamma_{L\_op}\cdot(Vi_{LL})}$$

$$\boxed{RF_{shear\_Op} = 1.439}$$

At the Service III Limit State (Inventory Level):

$$RF = \frac{f_R - \gamma_D\cdot(f_D)}{\gamma_{servLL}(f_{LLIM})}$$

$T := ns\cdot A_s\cdot f_{pe}$ $\boxed{T = 1626}$ kips

$f_{pb} := \dfrac{T}{A_g} + \dfrac{T\cdot(e_s)}{S_b}$ $\boxed{f_{pb} = 4.414}$ ksi

Allowable Tensile Stress

$t_{all} := -0.19\cdot\sqrt{f'_c}$; $|\, t_{all} \,| \le 0.6$ ksi $\boxed{t_{all} = -0.537}$ ksi

$f_R := f_{pb} - t_{all}$ $\boxed{f_R = 4.951}$ ksi

Live Load Stresses:

$f_{LLIM} := \dfrac{M_{LLIM}\cdot 12}{S_{cgb}}$ $\boxed{f_{LLIM} = 1.496}$ ksi

Dead Load Stresses:

$f_{DL} := \dfrac{M_{DC1}\cdot 12}{S_b} + \dfrac{M_{DC2}\cdot 12}{S_{cgb}}$ $\boxed{f_{DL} = 3.240}$ ksi

$RF_{serviceIII} := \dfrac{f_R - 1.0\cdot(f_{DL})}{\gamma_{servLL}\cdot(f_{LLIM})}$ $\boxed{RF_{serviceIII} = 1.430}$

E45-2.10 Legal Load Rating

Since the Operating Design Load Rating RF>1.0, the Legal Load Rating is not required. The Legal Load computations that follow have been done for illustrative purposes only. Shear ratings have not been illustrated.

Live Loads used will be the AASHTO Legal Loads per Figure 45.4-1 and AASHTO Specialized Hauling Vehicles per Figure 45.4-2.

$\boxed{g_i = 0.636}$

$IM := 33$ %

* WisDOT does not allow for a dynamic load allowance reduction based on the smoothness of the roadway surface. Thus, IM=33%

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Live Load Factors taken from Tables 45.3-1 and 45.3-2

$\phi_c := 1.0$ $\phi_s := 1.0$

$\phi := 1.0$

$\gamma_{L\_Legal} := 1.45$ $\gamma_{DC} := 1.25$

$\gamma_{L\_SU} := 1.45$

For Flexure

$$RF_{Legal} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_Legal} \cdot (M_{LLIM})}$$

$$RF_{SU} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_SU} \cdot (M_{LLIM})}$$

$E_s := 29000$	ksi, Modulus of Elasticity of the reinforcing steel
$w_p := 0.387$	weight of Wisconsin Type LF parapet, klf
$t_s := 8$	slab thickness, in
$t_{se} := 7.5$	effective slab thickness, in
$skew := 0$	skew angle, degrees
$w_c := 0.150$	kcf
$h := 2$	height of haunch, inches

E45-3.2 Modulus of Elasticity of Beam and Deck Material

Based on past experience, the modulus of elasticity for the precast and deck concrete are given in Chapter 19 as $E_{beam6} := 5500$ ksi and $E_{deck4} := 4125$ ksi for concrete strengths of 6 and 4 ksi respectively. The values of E for different concrete strengths are calculated as follows (ksi):

$$E_{beam8} := 5500 \cdot \frac{\sqrt{f'_c \cdot 1000}}{\sqrt{6000}} \qquad \boxed{E_{beam8} = 6351} \qquad E_B := E_{beam8}$$

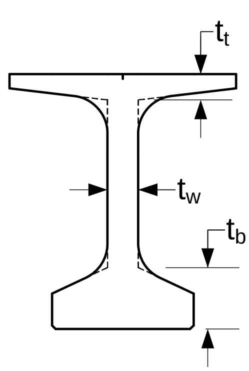
$$E_D := E_{deck4}$$

$$n := \frac{E_B}{E_D} \qquad \boxed{n = 1.540}$$

E45-3.3 Section Properties

54W Girder Properties:

$w_{tf} := 48$	in
$t_w := 6.5$	in
$ht := 54$	in
$b_w := 30$	width of bottom flange, in
$A_g := 798$	in^2
$I_g := 321049$	in^4
$y_t := 27.70$	in
$y_b := -26.30$	in



E45-3.4 Girder Layout

$S := 7.5$ Girder Spacing, feet

$s_{oh} := 2.50$ Deck overhang, feet

$ng := 6$ Number of girders

E45-3.5 Loads

$w_g := 0.831$ weight of 54W girders, klf

$w_d := 0.100$ weight of 8-inch deck slab (interior), ksf

$w_h := 0.100$ weight of 2-in haunch, klf

$w_{di} := 0.410$ weight of each diaphragm on interior girder (assume 2), kips

$w_{ws} := 0.020$ future wearing surface, ksf

$w_p = 0.387$ weight of parapet, klf

E45-3.5.1 Dead Loads

Dead load on non-composite (DC):

interior:

$$w_{dlii} := w_g + w_d \cdot S + w_h + 2 \cdot \frac{w_{di}}{L} \qquad \boxed{w_{dlii} = 1.687} \text{ klf}$$

* Dead load on composite (DC):

$$w_p := \frac{2 \cdot w_p}{ng} \qquad \boxed{w_p = 0.129} \text{ klf}$$

* Wearing Surface (DW):

$$w_{ws} := \frac{w \cdot w_{ws}}{ng} \qquad \boxed{w_{ws} = 0.133} \text{ klf}$$

* **LRFD [4.6.2.2.1]** states that permanent loads on the deck may be distributed uniformly among the beams. This method is used for the parapet and future wearing surface loads.

Calculate the composite girder section properties:

effective slab thickness; $\boxed{t_{se} = 7.50}$ in

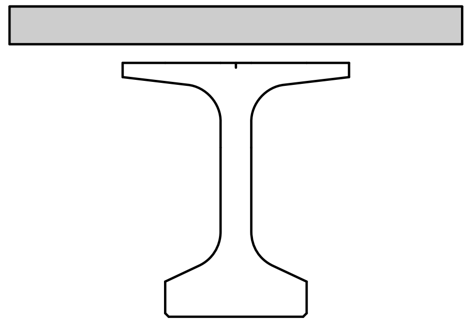
effective slab width; $\boxed{w_{eadj} = 58.46}$ in

haunch thickness; $\boxed{h = 2.0}$ in

total height; $h_c := ht + h + t_{se}$

$\boxed{h_c = 63.50}$ in

$\boxed{n = 1.540}$



Note: The area of the concrete haunch is not included in the calculation of the composite section properties.

Component	Ycg	A	AY	AY^2	I	I+AY^2
Deck	59.75	438	26197	1565294	2055	1567349
Girder	26.3	798	20987	551969	321049	873018
Haunch	55	0	0	0	0	0
Summation		1236	47185			2440367

$\Sigma A := 1236$ in^2

$\Sigma AY := 47185$ in^4

$\Sigma IplusAYsq := 2440367$ in^4

$y_{cgb} := \frac{-\Sigma AY}{\Sigma A}$ $\boxed{y_{cgb} = -38.2}$ in

$y_{cgt} := ht + y_{cgb}$ $\boxed{y_{cgt} = 15.8}$ in

$A_{cg} := \Sigma A$ in^2

$I_{cg} := \Sigma IplusAYsq - A_{cg} \cdot y_{cgb}^{\ 2}$ $\boxed{I_{cg} = 639053}$ in^4

Deck:

$$S_c := n \cdot \frac{I_{cg}}{y_{cgt} + h + t_{se}}$$ $\boxed{S_c = 38851}$ in^4

E45-3.11 Flexural Strength Capacity at Pier

All of the continuity reinforcement is placed in the top mat. Therefore the effective depth of the section at the pier is:

$cover := 2.5$ in

$bar_{trans} := 5$ (transverse bar size)

$Bar_D(bar_{trans}) = 0.625$ in (transverse bar diameter)

$Bar_{No} = 10$

$Bar_D(Bar_{No}) = 1.27$ in (Assumed bar size)

$$d_e := ht + h + t_s - cover - Bar_D(bar_{trans}) - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_e = 60.24}$ in

For flexure in non-prestressed concrete, $\phi_f := 0.9$.

The width of the bottom flange of the girder, $b_w = 30.00$ inches.

The continuity reinforcement is distributed over the effective flange width calculated earlier, $w_e = 90.00$ inches.

From E19-2, use a longitudinal bar spacing of #4 bars at $s_{longit} := 8.5$ inches. The continuity reinforcement is placed at 1/2 of this bar spacing, .

#10 bars at 4.25 inch spacing provides an $\boxed{As_{prov} = 3.57}$ in^2/ft, or the total area of steel provided:

$$As := As_{prov} \cdot \frac{w_e}{12}$$ $\boxed{As = 26.80}$ in^2

Calculate the capacity of the section in flexure at the pier:

Check the depth of the compression block:

$\alpha_1 := 0.85$ (for $f'_c \le 10.0$ ksi) **LRFD [5.7.2.2]**

$$a := \frac{As \cdot f_y}{\alpha_1 \cdot b_w \cdot f'_c}$$ $\boxed{a = 7.883}$ in

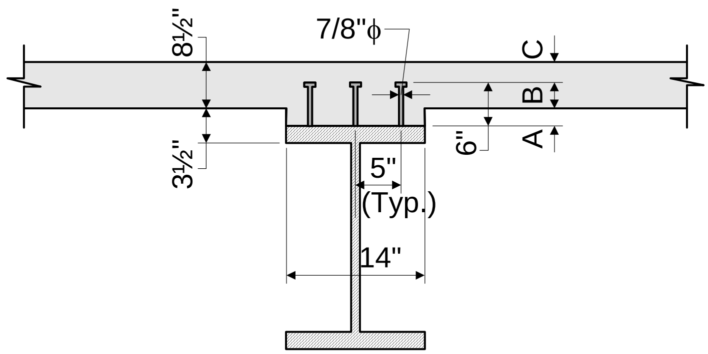


Figure E45-4.1-5

Composite Cross Section at Location of Maximum Positive Moment (0.4L)
(Note: 1/2" Intergral Wearing Surface has been removed for structural calcs.)

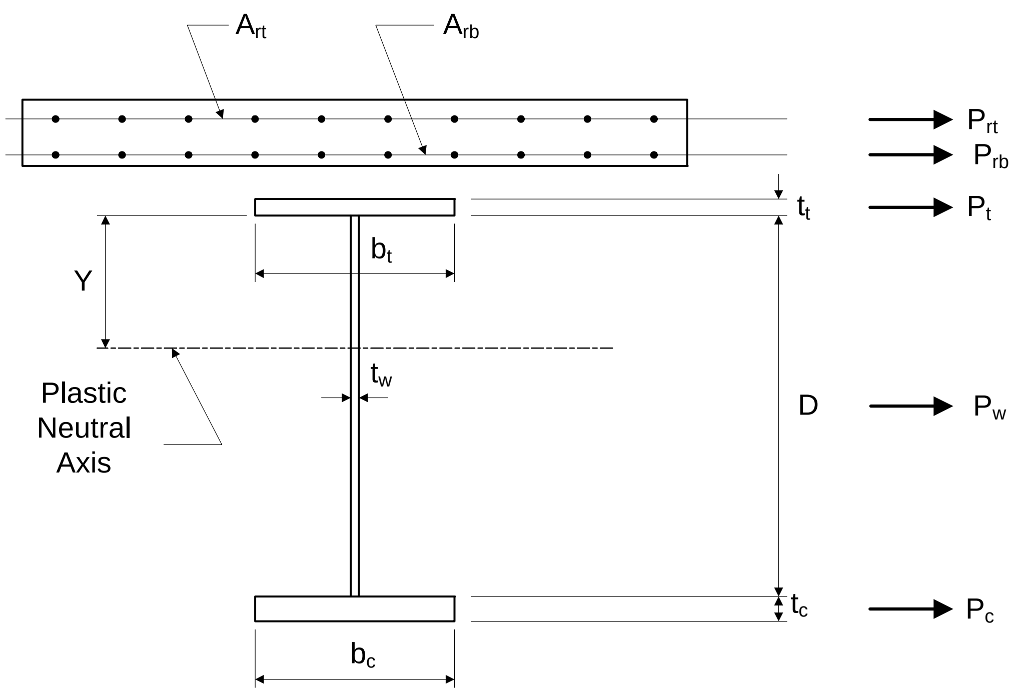


Figure E45-4.1-6

Composite Cross Section at Location of Maximum Negative Moment over Pier

$D := 54$ in

$t_W := 0.5$ in

E45-4.2 Compute Section Properties

Since the superstructure is composite, several sets of section properties must be computed **LRFD [6.10.1.1]**. The initial dead loads (or the noncomposite dead loads) are applied to the girder-only section. For permanent loads assumed to be applied to the long-term composite section, the long-term modular ratio of 3n is used to transform the concrete deck area **LRFD [6.10.1.1.1b]**. For transient loads assumed applied to the short-term composite section, the short-term modular ratio of n is used to transform the concrete deck area.

The modular ratio, n, is computed as follows:

$$n = \frac{E_s}{E_c}$$

Where:

E_s = Modulus of elasticity of steel (ksi)

E_c = Modulus of elasticity of concrete (ksi)

$E_s = 29000.00$ ksi **LRFD [6.4.1]**

$E_c = 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ **LRFD [C5.4.2.4]**

Where:

K_1 = Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction

w_c = Unit weight of concrete (kcf)

f'_c = Specified compressive strength of concrete (ksi)

$w_c = 0.15$ kcf **LRFD [Table 3.5.1-1 & C3.5.1]**

$f'_c = 4.00$ ksi **LRFD [Table 5.4.2.1-1 & 5.4.2.1]**

$K_1 := 1.0$ **LRFD [5.4.2.4]**

$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ $\boxed{E_c = 3834}$ ksi

$n := \frac{E_s}{E_c}$ $\boxed{n = 7.6}$ **LRFD [6.10.1.1.1b]**

Therefore, use: $n := 8$