DISCLAIMER

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

1.1 Introduction

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

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

1.2 Index

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2.1 Organizational Charts

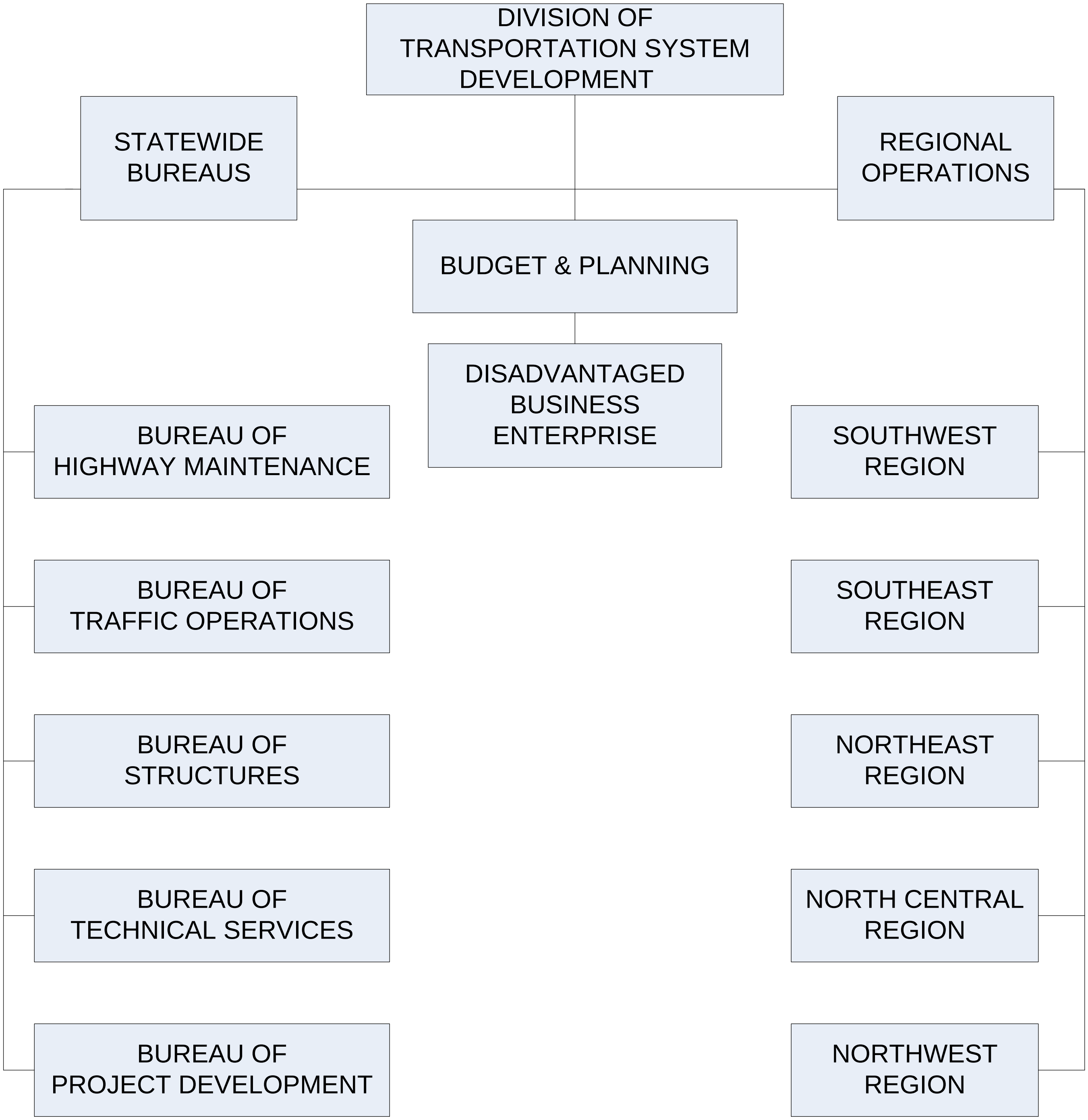


Figure 2.1-1
Division of Transportation System Development

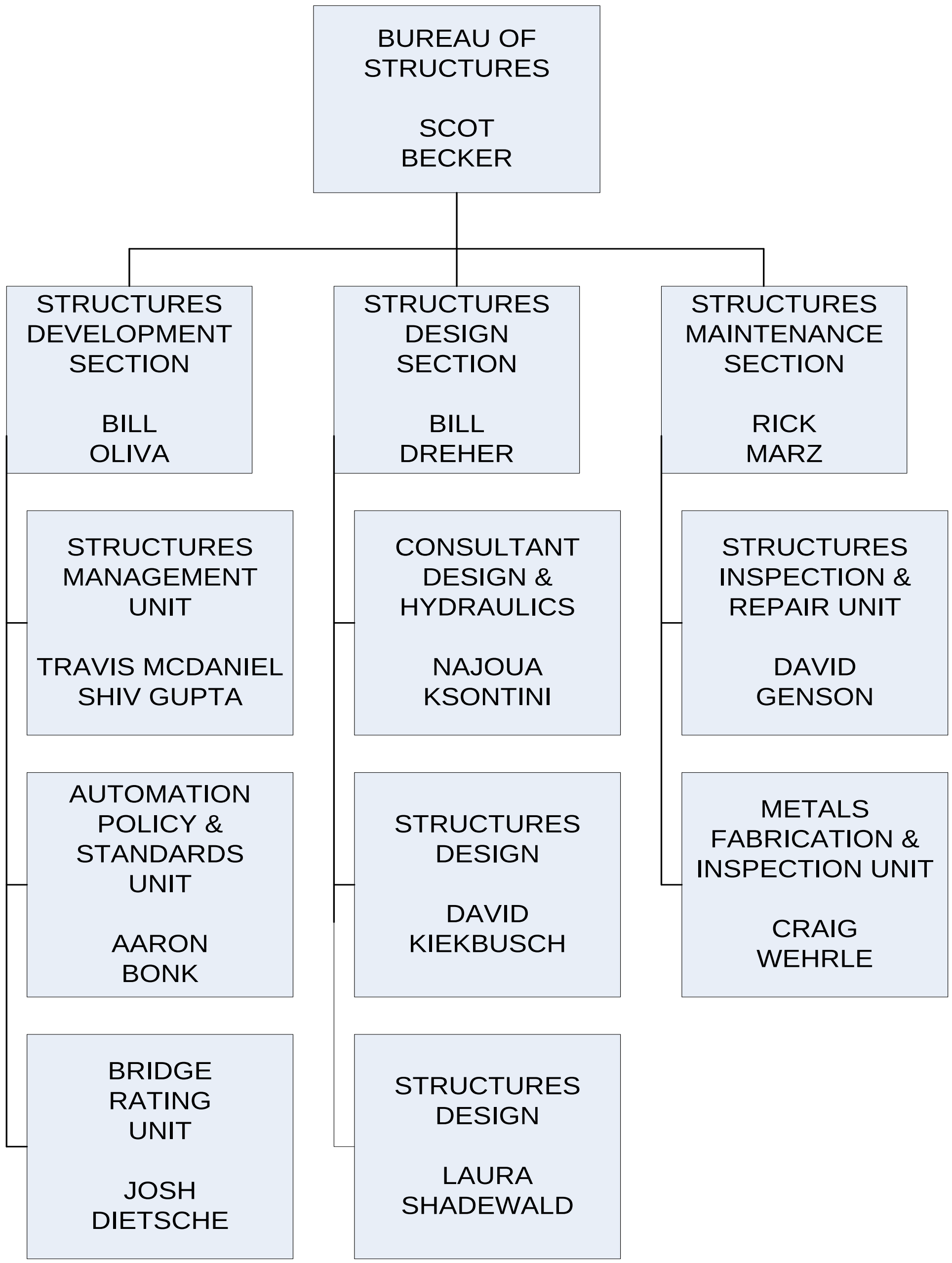
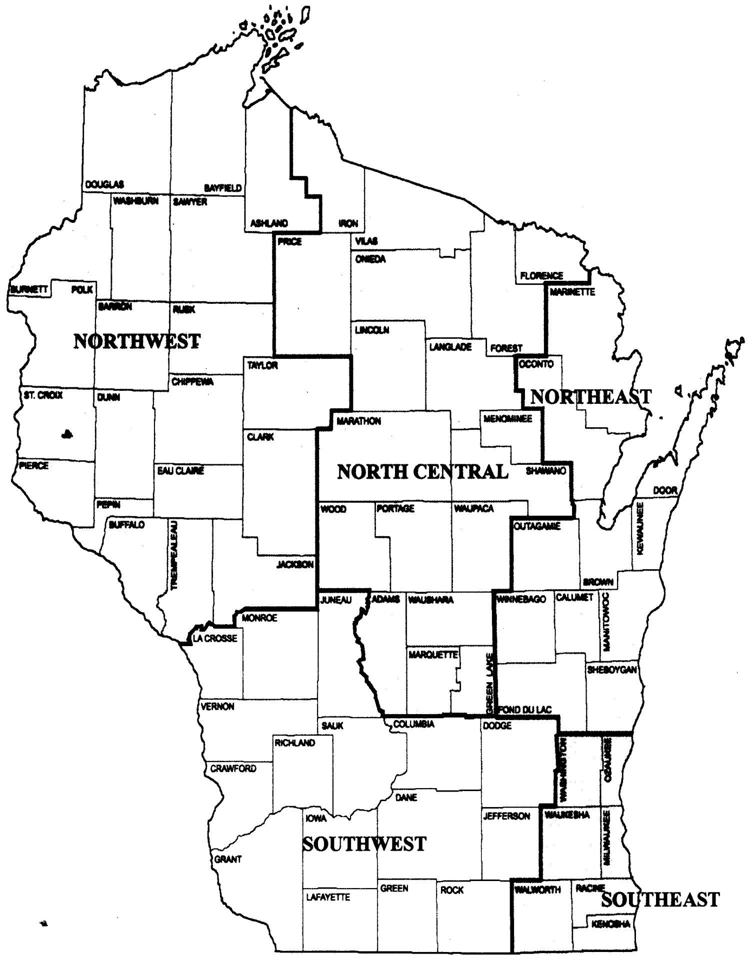


Figure 2.1-2
Bureau of Structures



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

2.4 Bridge Standards and Insert Sheets

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

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

1. FHWA Approval of Structure Standards Process

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

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

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

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

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

2.5 Bridge Numbers

An official number referred to as the Bridge Number is assigned to every structure on the State Highway System for the purpose of having a definite designation. The Bridge Number is hyphenated with the first digit being either a B, C, P, S, R, M or N. B is assigned to all structures over 20 ft. in length, including culverts. C is assigned to all structures 20 ft. or less in length but must have a cross-sectional area greater than 25 square feet. Do not include pipes that do not require structural computations. P designates structures for which there are no structural plans on file. S is for sign structures, R is for Retaining Walls, and N is for noise barriers. M is for miscellaneous structures where it is desirable to have a plan record. Bridges on state boundary lines also have a number designated by the adjacent state.

WisDOT Policy Item:

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

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

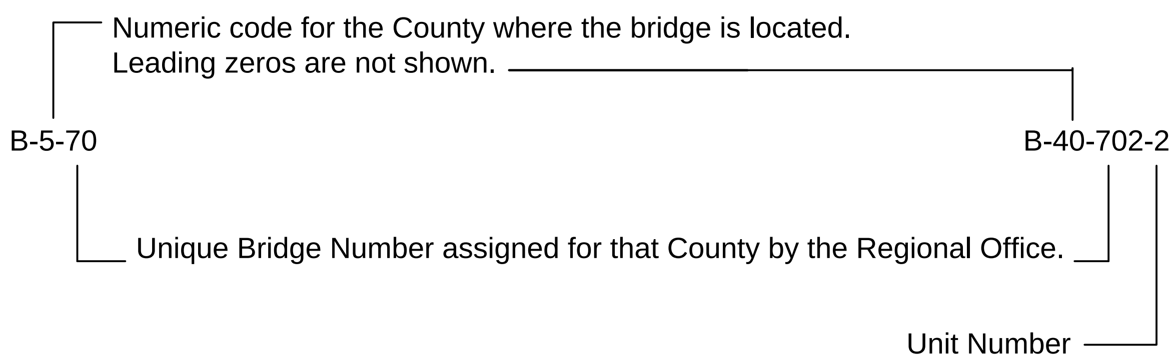


Figure 2.5-1
Bridge Number Detail

A set of nested pipes may be given a Bridge Number if the distance between the outside walls of the end pipes exceeds 20 ft. and the clear distance between pipe openings is less than half the diameter of the smallest pipe.

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

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

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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 much 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.

4.2 General Aesthetic Guidelines

Primary features – in relative order of importance:

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

Secondary features – in relative order of importance:

- Color
- Pattern and texture
- Ornamentation

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

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

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

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

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 or stopped traffic is beneath a bridge, formliners or other aesthetic enhancements are not viewable 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.

Grade and/or Skew

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

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

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

4.4 Secondary Features

Color

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

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

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

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

Pattern and Texture

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

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

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

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

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

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

Ornamentation

More is seldom better.

> "In bridge building… to overload a structure or any part thereof with ornaments… would be to suppress or disguise the main members and to exhibit an unbecoming wastefulness. The plain or elaborate character of an entire structure must not be contradicted by any of its parts."
>
> \- J.B. Johnson, 1912

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5.1 Factors Governing Bridge Costs

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

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

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

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

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

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
206.6000.S	Temporary Shoring	SF	16.90
210.0100	Backfill Structure	CY	20.59
303.0115	Pit Run	CY	16.84
311.0115	Breaker Run	CY	17.20
502.0100	Concrete Masonry Bridges	CY	501.80
502.1100	Concrete Masonry Seal	CY	186.28
502.2000	Compression Joint Sealer Preformed Elastomeric (width)	LF	26.08
502.3100	Expansion Device (structure) (LS)	LF	163.12
502.3110.S	Expansion Device Modular (structure) (LS)	LF	1,664.00
502.3200	Protective Surface Treatment	SY	2.35
502.6500	Protective Coating Clear	GAL	65.00
503.0128	Prestressed Girder Type I 28-Inch	LF	145.00
503.0136	Prestressed Girder Type I 36-Inch	LF	175.45
503.0137	Prestressed Girders Type I 36W-Inch	LF	160.40
503.0145	Prestressed Girder Type I 45-Inch	LF	182.66
503.0146	Prestressed Girders Type I 45W-Inch	LF	161.24
503.0154	Prestressed Girder Type I 54-Inch	LF	282.73
503.0155	Prestressed Girder Type I 54W-Inch	LF	178.94
503.0170	Prestressed Girder Type I 70-Inch	LF	--
503.0172	Prestressed Girders Type I 72W-Inch	LF	211.80
503.0182	Prestressed Girder Type I 82W-Inch	LF	--
504.0100	Concrete Masonry Culverts	CY	461.75
504.0500	Concrete Masonry Retaining Walls	CY	521.15
505.0405	Bar Steel Reinforcement HS Bridges	LB	0.87
505.0410	Bar Steel Reinforcement HS Culverts	LB	0.90
505.0415	Bar Steel Reinforcement HS Retaining Walls	LB	0.76
505.0605	Bar Steel Reinforcement HS Coated Bridges	LB	0.91
505.0610	Bar Steel Reinforcement HS Coated Culverts	LB	1.31
505.0615	Bar Steel Reinforcement HS Coated Retaining Walls	LB	0.99
506.0105	Structural Carbon Steel	LB	2.30
506.0605	Structural Steel HS	LB	1.43
506.2605	Bearing Pads Elastomeric Non-Laminated	EACH	71.95
506.2610	Bearing Pads Elastomeric Laminated	EACH	971.54
506.3005	Welded Stud Shear Connectors 7/8 x 4-Inch	EACH	4.50
506.3010	Welded Stud Shear Connectors 7/8 x 5-Inch	EACH	3.30
506.3015	Welded Stud Shear Connectors 7/8 x 6-Inch	EACH	3.18
506.3020	Welded Stud Shear Connectors 7/8 x 7-Inch	EACH	4.55
506.3025	Welded Stud Shear Connectors 7/8 x 8-Inch	EACH	3.33
506.4000	Steel Diaphragms (structure)	EACH	501.00
506.5000	Bearing Assemblies Fixed (structure)	EACH	838.90
506.6000	Bearing Assemblies Expansion (structure)	EACH	1,369.65
507.0200	Treated Lumber and Timber	MBM	7,416.00
508.1600	Piling Treated Timber Delivered	LF	56.00
00000	Preboring Cast-in-Place Concrete Piling	LF	99.30
550.2102	Piling CIP Concrete 10 ¾ x 0.219-Inch	LF	28.50

550.2104	Piling CIP Concrete 10 ¾ x 0.25-Inch	LF	29.07
550.2106	Piling CIP Concrete 10 ¾ x 0.365-Inch	LF	37.57
550.2108	Piling CIP Concrete 10 ¾ x 0.5-Inch	LF	--
550.2122	Piling CIP Concrete 12 ¾ x 0.219-Inch	LF	--
550.2124	Piling CIP Concrete 12 ¾ x 0.25-Inch	LF	35.49
550.2126	Piling CIP Concrete 12 ¾ x 0.375-Inch	LF	39.80
550.2128	Piling CIP Concrete 12 ¾ x 0.5-Inch	LF	65.00
550.2142	Piling CIP Concrete 14 x 0.219-Inch	LF	--
550.2144	Piling CIP Concrete 14 x 0.25-Inch	LF	--
550.2146	Piling CIP Concrete 14 x 0.375-Inch	LF	41.83
550.2148	Piling CIP Concrete 14 x 0.5-Inch	LF	--
550.2162	Piling CIP Concrete 16 x 0.219-Inch	LF	--
550.2164	Piling CIP Concrete 16 x 0.25-Inch	LF	--
550.2166	Piling CIP Concrete 16 x 0.375-Inch	LF	--
550.2168	Piling CIP Concrete 16 x 0.5-Inch	LF	--
550.1100	Piling Steel HP 10-Inch x 42 LB	LF	33.95
550.1120	Piling Steel HP 12-Inch x 53 LB	LF	36.98
550.1125	Piling Steel HP 12-Inch x 74 LB	LF	--
550.1140	Piling Steel HP 14-Inch x 73 LB	LF	46.00
550.0500	Pile Points	EACH	125.25
00000	Piling Steel Preboring	LF	101.20
512.1000	Piling Steel Sheet Temporary	SF	15.00
513.4050	Railing Tubular Type F (structure) (LS)	LF	125.20
513.4052 or 3	Railing Tubular Type F- (4 or 5) Modified (structure) (LS)	LF	138.42
513.4055	Railing Tubular Type H (structure) (LS)	LF	92.97
513.4060	Railing Tubular Type M (structure) (LS)	LF	169.63
513.4065	Railing Tubular Type PF (structure) (LS)	LF	--
513.4090	Railing Tubular Screening (structure) (LS)	LF	97.17
513.7050	Railing Type W (structure) (LS)	LF	75.29
SPV.0105	Parapet Concrete Type TX (LS) (estimate)	LF	160.00
00000	Parapet Concrete Type LF & A (estimate)	LF	80.00
513.7005	Railing Steel Type C1 (structure) (LS)	LF	71.39
513.7010	Railing Steel Type C2 (structure) (LS)	LF	149.28
513.7015	Railing Steel Type C3 (structure) (LS)	LF	108.25
513.7020	Railing Steel Type C4 (structure) (LS)	LF	89.16
513.7025	Railing Steel Type C5 (structure) (LS)	LF	--
513.7030	Railing Steel Type C6 (structure) (LS)	LF	107.76
514.0445	Floor Drains Type GC	EACH	2,064.45
514.2608	Downspout 8-Inch	LF	--
514.2625	Downspout 6-Inch	LF	100.00
516.0500	Rubberized Membrane Waterproofing	SY	27.19
517.1010.S	Concrete Staining (structure)	SF	1.12
517.1015.S	Concrete Staining Multi-Color (structure)	SF	2.80
517.1050.S	Architectural Surface Treatment (structure)	SF	5.05
604.0400	Slope Paving Concrete	SY	47.57
604.0500	Slope Paving Crushed Aggregate	SY	16.90
604.0600	Slope Paving Select Crushed Material	SY	22.62
606.0100	Riprap Light	CY	70.00
606.0200	Riprap Medium	CY	70.13
606.0300	Riprap Heavy	CY	44.82

606.0500	Grouted Riprap Light	CY	57.20
606.0600	Grouted Riprap Medium	CY	--
606.0700	Grouted Riprap Heavy	CY	78.17
612.0106	Pipe Underdrain 6-Inch	LF	5.11
612.0206	Pipe Underdrain Unperforated 6-Inch	LF	14.05
612.0406	Pipe Underdrain Wrapped 6-Inch	LF	6.19
616.0205	Fence Chain Link 5-FT	LF	21.69
616.0206	Fence Chain Link 6-FT	LF	--
616.0208	Fence Chain Link 8-FT	LF	--
645.0105	Geotextile Fabric Type C	SY	2.92
645.0111	Geotextile Fabric Type DF (Schedule A)	SY	2.42
645.0120	Geotextile Fabric Type HR	SY	2.94
652.0125	Conduit Rigid Metallic 2-Inch	LF	16.31
652.0135	Conduit Rigid Metallic 3-Inch	LF	23.69
652.0225	Conduit Rigid Nonmetallic Schedule 40 2-Inch	LF	5.50
652.0235	Conduit Rigid Nonmetallic Schedule 40 3-Inch	LF	4.88
SPV.0035	HPC Masonry Superstructure	CY	576.36
SPV.0085	Stainless Steel Reinforcement	LB	4.04
SPV.0165	Anti-Graffiti Coating	SF	1.42
SPV.0180	Pigmented Protective Surface Treatment	SY	6.08
SPV.0180	Protective Polymer Coating	SY	43.50

Table 5.3-1

Contract Unit Bid Prices for New Structures

Item No.	Bid Item	Unit	Cost
455.0___	Asphaltic Material _________	TON	50.00
460.1___	HMA Pavement Type _____	TON	48.62
502.5002	Masonry Anchors Type L No. 4 Bars	EACH	11.99
502.5005	Masonry Anchors Type L No. 5 Bars	EACH	16.87
502.5010	Masonry Anchors Type L No. 6 Bars	EACH	16.16
502.5015	Masonry Anchors Type L No. 7 Bars	EACH	16.53
502.5020	Masonry Anchors Type L No. 8 Bars	EACH	23.96
502.5025	Masonry Anchors Type L No. 9 Bars	EACH	26.00
502.6102	Masonry Anchors Type S 1/2-Inch	EACH	22.63
502.6105	Masonry Anchors Type S 5/8-Inch	EACH	15.49
502.6110	Masonry Anchors Type S 3/4-Inch	EACH	15.10
502.6115	Masonry Anchors Type S 7/8-Inch	EACH	--
502.6120	Masonry Anchors Type S 1-Inch	EACH	--
505.0904	Bar Couplers No. 4	EACH	11.73
505.0905	Bar Couplers No. 5	EACH	19.24
505.0906	Bar Couplers No. 6	EACH	11.70
505.0907	Bar Couplers No. 7	EACH	21.76
505.0908	Bar Couplers No. 8	EACH	39.26
505.0909	Bar Couplers No. 9	EACH	59.09
509.0301	Preparation Decks Type 1	SY	97.58
509.0302	Preparation Decks Type 2	SY	79.47
509.0500	Cleaning Decks	SY	8.25
509.1000	Joint Repair	SY	756.47
509.1200	Curb Repair	LF	38.96
509.1500	Concrete Surface Repair	SF	71.00
509.2000	Full-Depth Deck Repair	SY	456.60
509.2500	Concrete Masonry Overlay Decks	CY	514.50
509.5100.S	Polymer Overlay	SY	24.27

Table 5.3-2
Contract Unit Bid Prices for Rehab Structures

Table of Contents

6.1 Approvals, Distribution and Work Flow

Production of Structural Plans

Regional Office	Prepare Structure Survey Report.
Geotechnical Section (Bur. of Tech. Services)	Make site investigation and prepare Site Investigation Report. See 6.2.1 for exceptions.
Structures Development Sect. (Bur. of Structures)	Record Structure Survey Report.
Structures Design Section (Bur. of Structures)	Determine type of structure.
	Perform hydraulic analysis if required.
	Check roadway geometrics and vertical clearance.
	Review Site Investigation Report and determine foundation requirements. Check criteria for scour critical Bridges and record scour critical code on the preliminary plans.
	Draft preliminary plan layout of structure.
	Send copies of preliminary plans to Regional Office.
	If a railroad is involved, send copies of preliminary plans to the Rails & Harbors Section (Bureau of Transit, Local Roads, Rails & Harbors) who will forward details and information to the railroad company.
	If Federal aid funding is involved, send copies of preliminary plans to the Federal Highway Administration for major, moveable, and unusual bridges.
	If a navigable waterway is crossed, a Permit

	drawing to construct the bridge is sent to the Coast Guard. If FHWA determines that a Coast Guard permit is needed, send a Permit drawing to the Coast Guard. If Federal aid is involved, preliminary plans are sent to the Federal Highway Administration for approval.
	Review Regional Office comments and other agency comments, modify preliminary plans as necessary.
	Review and record project for final structural plan preparation.
	Assign project to a Structures Design Unit.
Structures Design Units (Bur. of Structures)	Prior to starting project, Designer contacts Regional Office to verify preliminary structure geometry, alignment, width and the presence of utilities.
	Prepare and complete design and final plans for the specified structure.
	Give completed job to Manager of Structures Design Section.
Manager, Structures Design Section (Bur. of Structures)	Review final structural plans.
	Review and revise or write special provisions as needed.
	Send copies of final structural plans and special provisions to Regional Offices.
	If a railroad is involved, send copies of final plans to the Rails & Harbors Section.
	Sign lead structural plan sheet.
	Deliver final structural plans and special

	provisions to the Bureau of Project Development.
Bur. of Project Development	Prepare final approved structural plans for pre- contract administration.

A map of navigable waterways in Wisconsin as defined by the Coast Guard is kept in the Consultant Design and Hydraulics Unit (Bureau of Structures).

6.2 Preliminary Plans

6.2.1 Structure Survey Report

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

Under the "Plans and Projects" heading:

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

The front of the form lists the supplemental information to be included with the report. Duplicate reports and supplemental information are required for Federal aid primary and Interstate projects.

6.2.1.1 BOS-Designed Structures

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

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

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

Under this process, the scheduling of geotechnical work is coordinated with the Bureau of Structures toward completion of the bridge plans by the final plan due date. If other geotechnical work is required for the project, the Project Manager should coordinate with the Regional Soils Engineer and the Geotechnical Section to promote efficiency of field drilling operations.

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

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

6.2.1.2 Consultant-Designed Structures

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

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

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

6.2.2 Preliminary Layout

6.2.2.1 General

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

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

6.2.2.2 Basic Considerations

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

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

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

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

3. Economics.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

6.2.2.3 Requirements of Drawing

6.2.2.3.1 Plan View

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

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

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

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

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

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

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

7. Detail the extent of slope paving or riprap.

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

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

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

11. Median width if dual highway.

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

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

14. Location of and vertical clearance at point of critical vertical clearance if highway or railroad separation. (For both roadway directions on divided highways).

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

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

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

18. Structure numbers on plan.

19. Excavation protection for railroads.

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

21. Location of deck lighting or utilities if any.

22. Name Plate location.

23. Bench Mark Cap Location

24. Locations of surface drains on approach pavement.

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

6.2.2.3.2 Elevation View

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

1. Profile of existing groundline or streambed.

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

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

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

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

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

7. Location and amount of minimum vertical clearance.

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

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

10. Location of fixed and expansion bearings.

11. Location and type of expansion devices.

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

13. Use a scale of 1” = 10’ whenever possible.

6.2.2.3.3 Cross-Section View

The cross-section view need only be a half section if symmetrical about a reference line, otherwise it is a full section taken normal to reference line. Use a scale of (1” = 4’) whenever possible. A view of a typical pier is shown as a part of the cross-section. The view shows the following general information:

1. Slab thickness, curb height and width, type of railing.

2. Horizontal dimensions tied into a reference line or centerline of roadway.
3. Steel beam or girder spacing with beam/girder depth.
4. For prestressed girders approximate position of exterior girders.
5. Direction and amount of crown or superelevation.
6. Point referred to on profile grade.
7. Type of pier with size and number of columns proposed.
8. For solid, hammerhead or other type pier approximate size to scale.
9. If length of concrete pier cap between outer pier columns exceeds approximately 60 feet, provide an opening in the cross girder for temperature changes and concrete shrinkage, or design the pier cap for temperature and shrinkage to eliminate the opening.
10. Dimension minimum depth of bottom of footings below ditch or finished ground line or if railroad crossing below top of rail.
11. Location for public and private utilities to be carried in the superstructure. Label owner's name of utilities.
12. Location of lighting on the deck or under the deck if any.

6.2.2.3.4 Other Requirements

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

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

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

   <u>Ultimate Stresses for Materials</u>:

   - Concrete Superstructure
   - Concrete Substructure
   - Bar Steel Reinforcement

- Structural Steel
- Prestressed Concrete
- Prestressing Steel

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

Foundations

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

Ratings (Plans Including Ratings that have been Changed)

Live Load:

Design Loading: HL-93

Inventory Rating Factor: RF = X.XX

Operating Rating Factor: RF = X.XX

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

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

Ratings (Plans Including Ratings that have not been Changed)

Live Load:

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

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

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

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

Hydraulic Data

Base Flood

- 100 Year Discharge
- Stream Velocity

- 100 Year Highwater Elevation
- Q_2 & Q_2 Elevation (Based on new structure opening)
- Waterway Area
- Drainage Area
- Scour Critical

Overtopping Flood OR (Overtopping N/A, for Floods > the 100 Year Flood)

- Overtopping Frequency
- Overtopping Elevation
- Overtopping Discharge

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

6.2.2.4 Utilities

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

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

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

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

6.2.3 Distribution of Exhibits

6.2.3.1 Federal Highway Administration (FHWA).

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

1. Projects on the Interstate System

2. Projects on the National Highway System (NHS) but not on the Interstate System, unless it is determined by FHWA and WisDOT that the responsibilities can be assumed by WisDOT

3. Projects on non-NHS Federal-aid highways, and eligible projects on public roads which are not Federal-aid highways if WisDOT determines that it is not appropriate for WisDOT to assume the responsibilities

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

Unusual bridges have the following characteristics:

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

Examples of unusual bridges:

- Cable-stayed
- Suspension
- Arch
- Segmental concrete
- Movable
- Truss
- Bridge types that deviate from AASHTO bridge design standards or AASHTO guide specifications for highway bridges
- Major bridges using load and resistance factor design specifications
- Bridges using a three-dimensional computer analysis
- Bridges with spans exceeding 350 feet

Examples of unusual structures:

- Tunnels

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

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

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

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

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

6.2.3.2 Coast Guard

Current permit application guides published by the 2nd or 9th Coast Guard District should be followed. For Federal Aid projects, applicants must furnish two copies of the Final Environmental Impact Statement accepted by the lead agency. The Regional Office will also forward Water Quality Certification obtained from the Department of Natural Resources.

6.2.3.3 Regions

One print of all preliminary drawings is sent to the Regional Office involved, for their review. For structures financed partially or wholly by a county, city, village or township, their approval should be obtained by the Regional Office and approval notice forwarded to the Bureau of Structures.

6.2.3.4 Utilities

For all structures which involve a railroad, four prints of the preliminary drawing are submitted to the Utilities & Access Management Unit for submission to the railroad company for approval.

If private or public utilities wish to make application to attach their facilities (water, and sewer mains, ducts, cables, etc.) to the structure, they must apply to the Utilities & Access Management Unit for approval.

6.2.3.5 Other Agencies

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

6.3 Final Plans

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

6.3.1 General Requirements

6.3.1.1 Drawing Size

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

6.3.1.2 Scale

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

6.3.1.3 Line Thickness

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

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

6.3.1.4 Lettering and Dimensions

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

6.3.1.5 Notes

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

6.3.1.6 Standard Insert Drawings

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

Standard insert sheets can be found at:
http://trust.dot.state.wi.us/extntgtwy/dtid_bos/extranet/structures/index.htm

6.3.1.7 Abbreviations

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

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

Discharge	DISCH.	Near Side, Far Side	N.S.F.S.
Per Cent	%	Sidewalk	SDWK.
Plate	PL	South	S.
Point of Curvature	P.C.	Space	SPA.
Point of Intersection	P.I.	Specification	SPEC
Point of Tangency	P.T.	Standard	STD.
Point on Curvature	P.O.C.	Station	STA.
Point on Tangent	P.O.T.	Structural	STR.
Property Line	P.L.	Substructure	SUBST.
Quantity	QUAN.	Superstructure	SUPER.
Radius	R.	Surface	SURF.
Railroad	R.R.	Superelevation	S.E.
Railway	RY.	Symmetrical	SYM
Reference	REF.	Tangent Line	TAN. LN.
Reinforcement	REINF.	Transit Line	T/L
Reinforced Concrete Culvert Pipe	R.C.C.P.	Transverse	TRAN.
Required	REQ'D.	Variable	VAR.
Right	RT.	Vertical	VERT.
Right Hand Forward	R.H.F.	Vertical Curve	V.C.
Right of Way	R/W	Volume	VOL.
Roadway	RDWY.	West	W.
Round	ø	Zinc Gauge	ZN. GA.
Section	SEC.		

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

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

6.3.2 Plan Sheets

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

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

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

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

See the BOS web page, CADD Resource Files, for the latest sheet boarders 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 will determine the factored bearing resistance by visual inspection prior to construction of the abutment footing.

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

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

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

8. Estimated Quantities

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

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

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

9. General Notes

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

10. List of Drawings

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

11. Bench Marks

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

12. Title Block

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

13. Professional Seal

    All final bridge plans prepared by Consultants or Governmental Agencies shall be professionally sealed, signed, and dated on the general plan sheet.

    This is not required for WisDOT prepared plans, as they are covered elsewhere.

6.3.2.1.1 Plan notes for New Bridge Construction

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

WisDOT policy item:

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

It is the responsibility of the design engineer to use original structure plans, as-built structure plans, shop drawings, field surveys and structure inspection reports as appropriate when producing rehabilitation structure plans. If uncertainty persists after reviewing available documentation, a field visit may be necessary by the design engineer.

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

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

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

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

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

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

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

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

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

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

6.3.2.2 Subsurface Exploration

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

1. Plan View

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

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

2. Elevation

   Show a centerline profile of existing ground elevation.

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

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

6.3.2.3 Abutments

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

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

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

1. Plan View

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

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

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

2. Elevation

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

    b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

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

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

Use as many sheets as necessary to show all details clearly. One sheet may show several piers if only the height, elevations and other minor details are different.

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

1. Plan View

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

2. Elevation

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

3. Footing Plan

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

4. Bar Steel Listing and Details

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

6. Cross Section thru Column and Pier Cap

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

6.3.2.5 Superstructure

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

6.3.2.5.1 All Structures

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.2.5.2 Steel Structures

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

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

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

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

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

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

6.3.2.5.3 Railing and Parapet Details

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

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

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

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

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

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

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

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

6.3.3.2 Box Culverts

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

1. Plan View
2. Longitudinal section
3. Section thru box
4. Wing elevations
5. Section thru wings
6. Section thru cutoff wall
7. Vertical construction joint
8. Bar steel clearance details
9. Header details

10. North point, Bench mark, Quantities

11. Bill of bars, Bar details

12. General notes, List of drawings, Rip rap layout

13. Inlet nose detail on multiple cell boxes

14. Corner details

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

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

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

6.3.3.3 Miscellaneous Structures

Detail plans for other structures such as retaining walls, sign bridges, pedestrian bridges, and erosion control structures are to be detailed with the same requirements as previously mentioned.

6.3.3.4 Standard Drawings

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

6.3.3.5 Insert Sheets

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

6.3.3.6 Change Orders and Maintenance Work

These plans are drawn on full size sheets. A Structure Survey Report should be submitted for all maintenance projects, including painting projects and polymer overlay projects. In addition to the SSR, final structure plans on standard sheet borders with the #8 tab should be submitted to BOS in the same fashion as other rehabilitation plans. Painting plans should include at minimum a plan view with overall width and length dimensions, the number of spans, an indication of the number and type of elements to be painted (girders, trusses, etc.), and an elevation view showing what the structure is crossing.

6.3.3.7 Name Plate and Bench Marks

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

6.3.4 Checking Plans

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

Give special attention to unique details and unusual construction problems. Take nothing for granted on the plans.

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

The Checker makes an independent Bill of Bars list to be sure the detailer has not omitted any bars when checking the quantity of bar steel.

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

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

After the plans are completed, the items in the survey folder are separated into the following groups by the Structures Design Unit Supervisor or plans checker:

6.3.4.1 Items to be Destroyed When Construction is Completed (Group A)

1. Miscellaneous correspondence and Transmittal letters
2. Preliminary drawings and computations
3. Prints of soil borings and plan profile sheets
4. Quantity computations and bill of bars
5. Shop steel quantity computations*
6. Design checker's computations
7. Designer Computations and computer runs of non-complex structures on non state maintained structures.
8. Layout sheets
9. Elevation runs and bridge geometrics
10. *Falsework plans*
11. Miscellaneous Test Report
12. Photographs of Bridge Rehabs

* These items are added to the packet during construction.

6.3.4.2 Items to be Destroyed when Plans are Completed (Group B)

1. All "void" material
2. All copies except one of preliminary drawings
3. Extra copies of plan and profile sheets
4. Preliminary computer design runs

Items in Group A should be placed together and labeled. Items in Group B should be discarded.

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

1. Structure Inventory Form (Available on DOTNET) - New Bridge File – Data for this form is completed by the preliminary designer and plans checker. It is submitted to the Structures Development Section for entry into the File.

2. Load Rating Input File - Permits File - The designers submit an electronic copy of the input data for load rating the structure to the Structures Development Section. It is located for internal use at //H32751/rating.

3. Designer Computations and Inventory Superstructure Design Run (Substructure computer runs as determined by the Engineer) - **HSI – The designers record design, inventory, operating ratings and maximum vehicle weights on the plans and place into the scanned folder.

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

5. Shop Drawings for Steel Bridges, Sign Bridges, Prestressed Girders, High Mast Poles, Retaining Walls, Floor Drains, Railings and all Steel Joints - HSI - Metals Fabrication & Inspection Unit or other source sends to the Structures Development Section to scan all data into HSI.

6. Mill Tests, Heat Numbers and Shop Inspection Reports for all Steel Main Members - HSI - Metals Fabrication & Inspection Unit sends electronic files data into HSI.

7. Hydraulic and Scour Computations, Contour Maps and Site Report - HSI - Data is placed into scanned folder by Consultant Design & Hydraulics Unit.

8. Subsurface Exploration Report - HSI - Report is placed into scanned folder by Consultant Design & Hydraulics Unit or electronic copies are loaded from Geotechnical files.

9. Structure Survey Report - HSI - Report is placed into scanned folder by Consultant Design & Hydraulics Unit.

10. As Built Plans - HSI - At bid letting, the printers place a digital image of plans in a computer folder and send to the Structures Development Section where the plan sheets are labeled and placed in HSI. As Built plans will replace bid letting plans when available and will be scanned by the Structures Development Section.

11. Inspection Reports - New Bridge File - The Structures Maintenance Section loads a copy of the following Inspection Reports into the New Bridge File.

Initial	Underwater (UW-Probe/Visual
Routine Visual	Movable
Fracture Critical	Damage

In-Depth	Interim
Underwater (UW)-Dive	Posted
Underwater (UW)-Surv	

Table 6.3-2
Various Inspection Reports

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

6.3.5 Processing Plans

1. Before P.S. & E. Process

   File plans in plan drawers by county for consultant work, or

   Maintain plans as PDF on E-plan server.

2. At P.S. & E. Processing

   Prepare plans for bid letting process.

3. After Structure Construction

   Any data in Design Folder is scanned and placed with bridge plans.

   Original plan sheets and Design Folders are discarded.

6.4 Computation of Quantities

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

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

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

Staged Construction - On projects where there is staged construction that will involve two construction seasons the following quantities should be split to match the staging to aid the contractor/fabricator: Concrete Masonry, Bar Steel Reinforcement, Structural Steel and Bar Couplers. The other items are not significant enough to justify separating.

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

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

6.4.1 Excavation for Structures Bridges (Structure)

This is a lump sum bid item. The limits of excavation are shown in the chapter in the manual which pertains to the structural item, abutments, piers, retaining walls, box culverts, etc.

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

6.4.2 Backfill Granular or Backfill Structure

Backfill Granular and Backfill Structure are bid in units of cubic yard. The pay limits and quantity computations of backfill at abutments are shown in Chapter 12 – Abutments.

6.4.3 Concrete Masonry Bridges

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

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

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

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

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

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

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

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

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

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

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

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

6.4.9 Piling Test Treated Timber (Structure)

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

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

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

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

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

6.4.11 Preboring CIP Concrete Piling or Steel Piling

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

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

Record the type, quantity is a Lump Sum.

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 or Floordrains Type H

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.

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

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

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

Estimate Type 2 Deck Preparation as 40% of Type 1 Deck Preparation. Record this quantity to the nearest square yard. Use 2" for depth of each Preparation, compute concrete quantity and add to Concrete Masonry Overlay Decks.

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard. Concrete quantity used, should be added to Concrete Masonry Overlay Decks.

6.4.27 Concrete Masonry Overlay Decks

Record this quantity to the nearest cubic yard. Estimate the quantity by using a thickness measured from the existing ground concrete surface to the plan gradeline. Calculate the minimum overlay thickness and add ½" for variations in the deck surface. Provide this average thickness on the plan, as well. Usually 1" of deck surface is removed by grinding.

6.4.28 Removing Old Structure STA. XX + XX.XX

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

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

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

6.4.30 Steel Diaphragms (Structure)

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

6.4.31 Welded Stud Shear Connectors X -Inch

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

6.4.32 Concrete Masonry Seal

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

6.4.33 Geotextile Fabric Type

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

6.4.34 Masonry Anchors Type L No. Bars

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

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

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

6.4.36 Piling Steel Sheet Temporary

This quantity is used when the designer determines that retention of earth is necessary during excavation and soil forces require the design of steel sheet piling.

Record this quantity to the nearest square foot for the area below the retained grade and one foot above the retained grade.

Following is a list of commonly used STSP’s and Bureau of Structures Special Provisions.

6.4.37 Temporary Shoring

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

Bid as square foot of exposed surface as shown on the plans.

6.4.38 Concrete Masonry Deck Patching

(Deck preparation areas) x 2” deck thickness.

6.4.39 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per S.Y. 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.

6.5 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

	code on preliminary plans. Refer to Chapter 8, Appendix 8-D.
	If a navigable waterway is crossed, complete necessary Coast Guard coordination.
	Submit preliminary plans and documents via e-submit for review and processing.
Structures Design Section	Record Bridge and project numbers.
	Review hydraulics for Stream Crossings.
	Review Preliminary Plan. A minimum of 30 days to review preliminary plans should be expected.
	If a railroad is involved, send a copy of preliminary plans to the Rails & Harbors Section.
	For special structure types (lift or moveable bridges; cost greater than $10,000,000), send preliminary plans to Federal Highway Administration for approval.
	Return preliminary plans and comments from Structures Design Section and other appropriate agencies to Consultant with a copy to the Regional Office.
	Forward Preliminary Plan and Hydraulic Data to DNR.
Consultant	Modify preliminary plan as required.
	Prepare and complete final design and plans for the specified structure.
	Write special provisions.
	At least **two months** in advance of the PS&E date, submit the following via e-submit: final plans, special provisions, computations, quantities, QA/QC Verification Sheet, Inventory Data Sheet, Bridge Load Rating Summary Form, LRFD Input File (Excel ratings spreadsheet).
Structures Design Section	Determine which final plans will be reviewed and perform review as applicable.
	If a railroad is involved, send a copy of final plans to Rails & Harbors Section.
	For special structure types (lift or moveable bridges; cost greater than $10,000,000), send final plans to Federal Highway Administration.
	For final plans that are reviewed, return comments to Consultant and send copy to Regional Office.
Consultant	Modify final plans and specifications as required.
	Submit modified final plans via e-submit as required.

Structures Design Section	Review modified final plans as applicable.
	Sign final plans.
Bureau of Project Development	Prepare final approved bridge plans for pre-development contract administration.

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Consultant Preliminary Plan Requirements

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

1. Structure Survey Report.
2. Preliminary Drawings.
3. Log Borings shown on the Subsurface Exploration Drawing which must be submitted now and can be included with the Final Plans.
4. Evaluation Report of Borings.
5. Contour Map.
6. Typical Section for Roadway Approaches.
7. Plan and Profile of Approach Roadways.
8. Hydraulic Report (see Chapter 8 - Hydraulics) is required for Stream Crossing Structures.
9. County Map showing Location of New and/or Existing Structures.
10. Any other information or Drawings which may influence Location, Layout or Design of Structure.

The above information is also required for Box Culverts except that a separate preliminary drawing is usually not prepared unless the Box Culvert has large wings or other unique features.

The type of structure is usually determined by the local unit of government and the Regional Office. However, Bureau of Structures personnel review the structure type and may recommend that other types be considered. In this regard it is extremely important that preliminary designs be coordinated to avoid delays and unnecessary expense in plan preparation.

If the final approach roadways are unpaved, detail protective armor angles at the roadway ends of bridge decks/slabs as shown on the Standard for Strip Seal Cover Plate Details.

6.5.3 Final Plan Requirements

The guidelines and requirements for Final Plan preparation are given in 6.3. The following exhibits are included as part of the Final Plans:

1. Final Drawings

   For all highway structures provide the maximum vehicle weight that can be safely carried based on the procedure and vehicle configuration provided in Chapter - Bridge Rating.

2. Design and Quantity Computations

   For all bridge structures, provide the analysis files used by the responsible engineer for determination of the controlling ratings.

3. Special Provisions covering unique items not in the Standard Specifications such as Electrical Equipment, New Proprietary Products, etc.

4. QA/QC Verification Sheet

5. Inventory Data Sheet, Bridge Load Rating Summary Form, LRFD Input File (Excel ratings spreadsheet).

On Federal or State Aid projects the contracts are let and awarded by the Wisconsin Department of Transportation. Shop drawing review and fabrication inspection are generally done by the Metals Fabrication and Inspection Unit. However, in some cases the consultant may check the shop drawings and an outside agency may inspect the fabrication. The Consultant contract specifies the scope of the work to be performed by the Consultant. Construction supervision and final acceptance of the project are provided by the State.

6.5.4 Design Aids & Specifications

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

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

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

Structure costs
Structure Special Provisions

http://www.dot.wisconsin.gov/business/engrserv/index.htm

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

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

http://bridges.transportation.org

http://www.arema.org

Table of Contents

7.1 Introduction

Disclaimer:

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

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

7.1.1 WisDOT ABC Initiative

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

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

7.1.2 ABC Overview

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

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

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

WisDOT policy item:

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

7.1.3 Accelerated Bridge Construction Technology

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

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

Table 7.1-1
ABC Terminology

7.1.4 ABC Methods

7.1.4.1 Prefabricated Bridge Elements

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

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

Figure 7.1-1
Prefabricated Pier Cap

Figure 7.1-2
Prefabricated Abutment

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

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

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

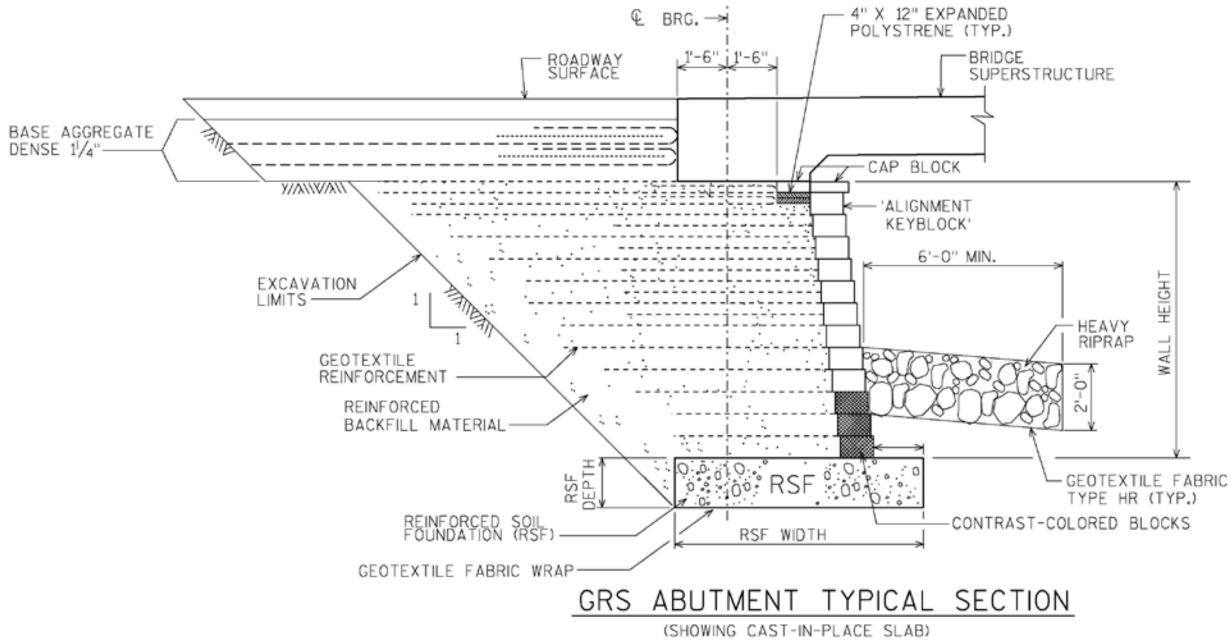

Figure 7.1-3
GRS-IBS Typical Cross Section

Figure 7.1-4
GRS-IBS Structure

Figure 7.1-5
GRS Abutment Layer During Construction

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

7.1.4.2.2 Application

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

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

2. Water velocities are less than 5 ft/s

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

4. Soil conditions permit shallow foundations.

5. Low-volume roadways

6. Single span structure with a span length less than 90 feet

7. Abutment wall height less than 22 feet (measured at the maximum wall height, from the top of the RSF to the top of the wall)

8. Wingwalls are parallel to roadway

9. Maximum skew angle of 15°

10. Short and long term settlements are tolerable

11. Differential settlement along the length of the abutment is tolerable to avoid twisting of the superstructure

12. Suitable construction materials available

7.1.4.2.3 Design Considerations

7.1.4.2.3.1 Hydraulics

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

WisDOT policy item:

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

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

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

7.1.4.2.3.2 Reinforced Soil Foundation (RSF) and Reinforced Soil Mass

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

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

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

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

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

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

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

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

7.1.4.2.3.3 Superstructure

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

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

7.1.4.2.3.4 Approach Integration

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

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

7.1.4.2.3.5 Design Details

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

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

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

7.1.4.3 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 ttemporary 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 tthe cantilevered portion and negative forces induced on the superstructure. During the lifting of the superstructure off the temporary supports, the bridge undergoes a stress reversal. When the girders are placed and the deck is poured, the girders deflect under the wet concrete weight, inducing stresses in the girder. When the deck is cured, the stresses in the girders induced by the deck are locked in, and the superstructure is in a state of equilibrium. Changing the support locations causes a stress reversal in the superstructure, which must be considered in the design of the bridge.

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

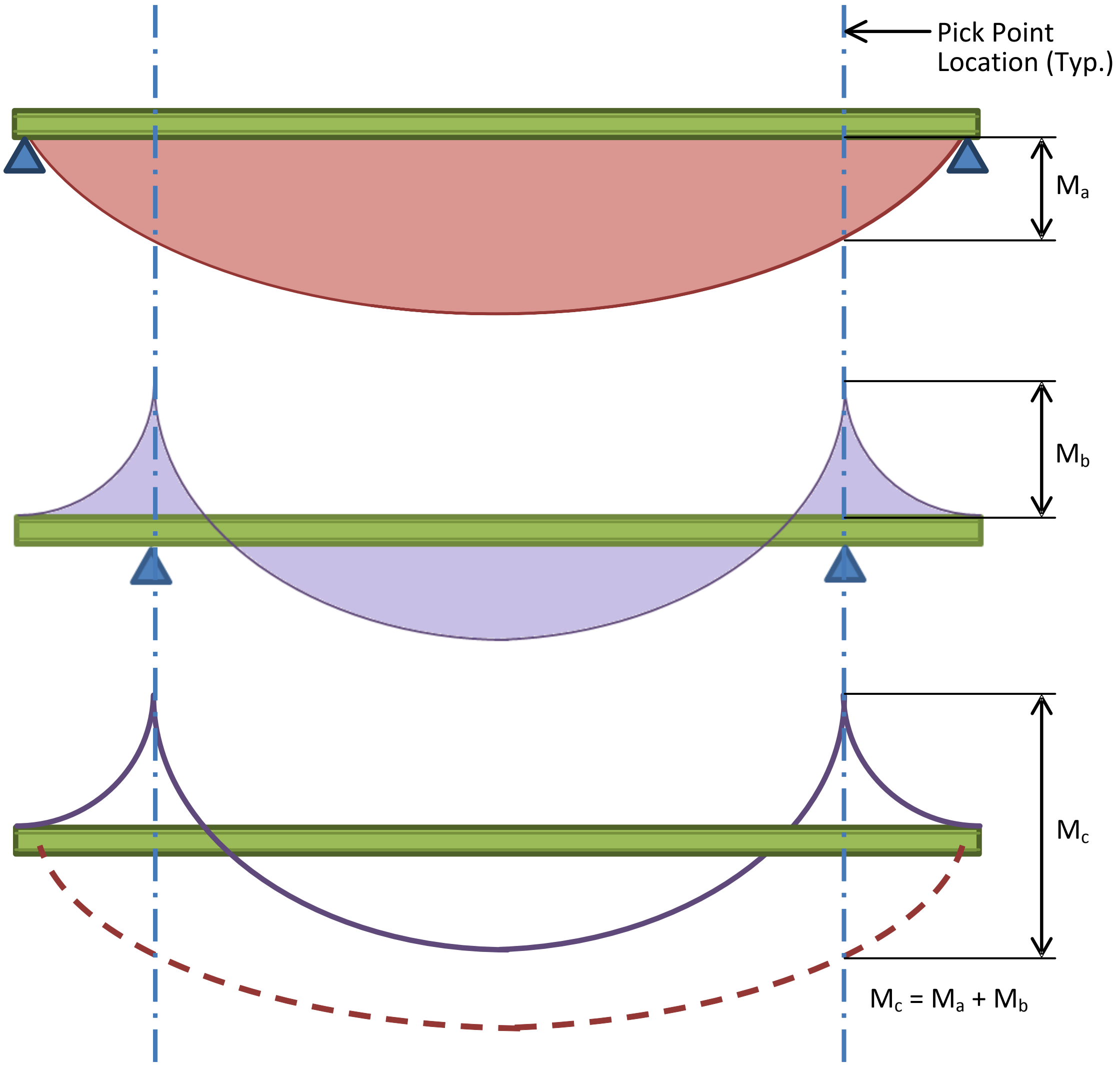

Figure 7.1-9
Support Change Moment Diagram (Illustrating Stress Reversal)

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

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

7.1.4.4.6.5 Deflection and Twist

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

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

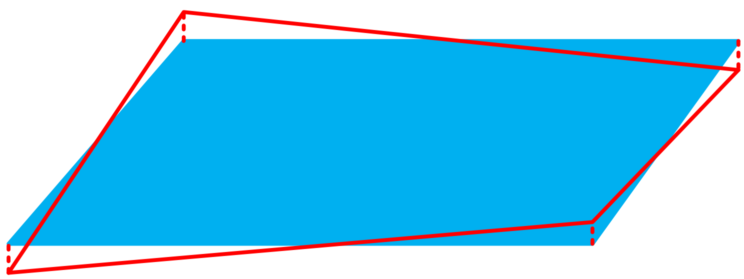

Figure 7.1-10
Bridge Warping Diagram

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

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

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

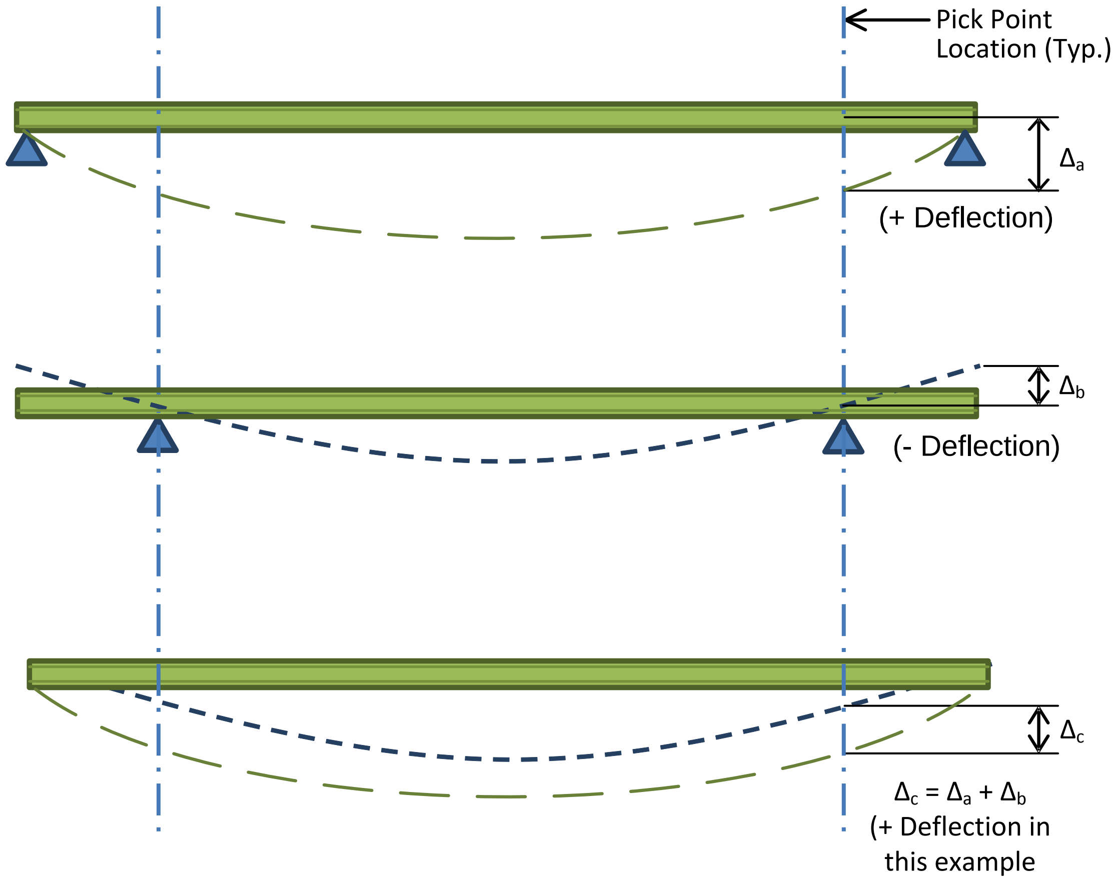

Figure 7.1-11
Support Change Deflection Diagram

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

7.1.4.4.7 Structure Removal Using SPMT

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

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

7.1.5 Project Delivery Methods/Bidding Process

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

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

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

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

WisDOT policy item:

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

7.2 ABC Decision-Making Guidance

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

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

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

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

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

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

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

Figure 7.2-1
ABC Decision-Making Matrix

7.2.1 Descriptions of Terms in ABC Decision-Making Matrix

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

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

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

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

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

Table 7.2-1
ABC Decision-Making Matrix Terms

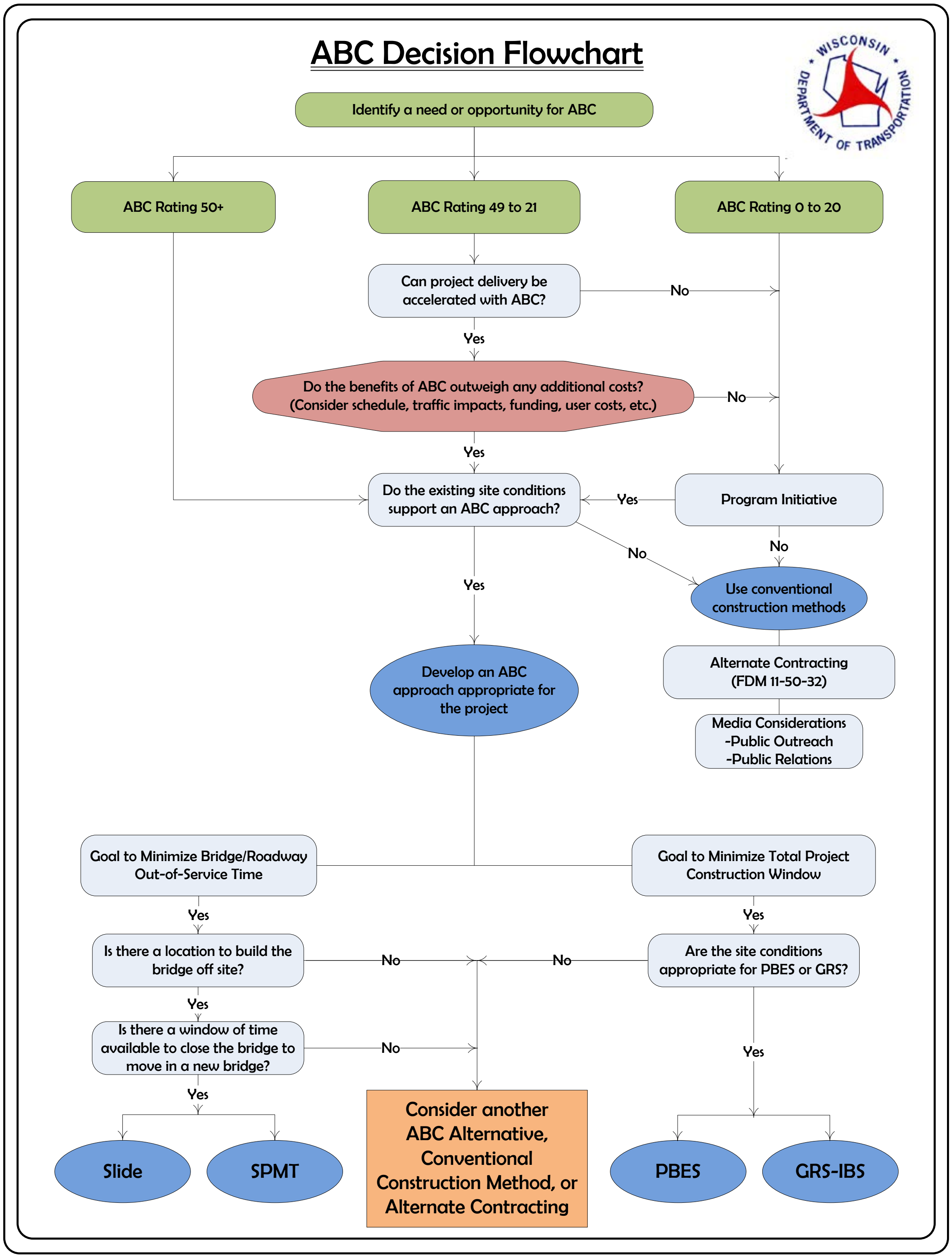

Figure 7.2-2
ABC Decision-Making Flowchart

7.3 References

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

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

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

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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.atwoodsystems.com/materials.

The *Wisconsin Construction and Materials Manual* (CMM) contains a description of procedures for material testing and acceptance requirements in Chapter 45, Section 25. Materials, unless otherwise permitted by the specifications, cannot be incorporated in the project until tested and approved by the proper authority.

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 limited to lengths of approximately 30 feet. The location of mandatory 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 (stirrups and ties). Dimensions for the bending details are shown as out to out of bar, as stated in the *Standard Specifications* Section 505.3.2. The diameter of a bend, measured on the inside of the bar for a standard bend is specified in **LRFD [5.10.2.3]**. Where a larger bend radius is required (non-standard bend) show the inside bend radius on the bar detail. When computing total bar lengths account for the accumulation in length in the bends. Use the figures mentioned above to account for accumulation in length for standard hooks and bends. One leg of bent bars is not dimensioned so that the tolerance for an error in the length due to bending is placed there. Fabrication tolerances for bent bars are specified in the *Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practices* or the *American Concrete Institute (ACI) Detailing Manual* as stated in Section 505.2.1 of the *Standard Specifications*.

Figure 9.3-1 shows typical detailing procedures for bars with bends.

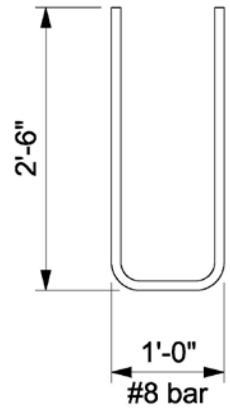

Figure 9.3-1
Bar Bend Detail (#8 bar)

Bar length = 1.0 ft + (2)(2.5 ft) – (2)(0.21 ft) = 5.58 ft or 5'-7" (to the nearest inch)

Where (0.21 ft) is (2.5"/12) and is the standard bar bend deduction found in Figure 9.9-1 for a #8 bar bent 90°.

9.3.3 Bill of Bars

Figure 9.9-3 shows a sample Bill of Bars table for a concrete slab. Different bar letter designations are used for abutments, slabs, and culverts, etc. If bundled bars are used, place a symbol adjacent to the bar mark of the bundled bars and a note below the Bill of Bars table stating the symbol represents bars to be bundled. A column for Bar Series is included if bars are cut.

9.3.4 Bar Series

A Bar Series table enables the detailer to detail bar steel in the simplest manner if it is used properly. Also, it helps the fabricator to prepare the Bill of Bars table.

The following general rules apply to the Bar Series table:

- Equal spacing of bars is required.
- There may be more than 1 Series with same number of bars.
- The total length of a bar is 60 feet (maximum).
- The minimum number of bars per Series is 4.

- Bent bars are bent after cutting.
- Set numbers are assigned to each Series used.

Figure 9.9-4 provides a sample layout for a Bar Series table. The Bill of Bars table will show the number of bars and the average bar length in the Series.

9.9 Bar Tables and Figures

(f'_c = 3500 psi; f_y = 60 ksi)											
BAR SPACING	BAR SIZE		4	5	6	7	8	9	10	11	TYPE
6" OR MORE	CLASS A	TOP [1]	1-2 1-5	1-5 1-9	1-9 2-1	2-3 2-9	3-0 3-8	3-9 4-7	4-10 5-10	5-11 7-2	UNCOATED EPOXY
	$1.0\ \ell_d$	OTHERS	1-0 1-0	1-0 1-6	1-3 1-10	1-8 2-5	2-2 3-3	2-9 4-1	3-5 5-2	4-3 6-4	UNCOATED EPOXY
	CLASS B	TOP [1]	1-6 1-9	1-10 2-3	2-3 2-8	3-0 3-7	3-11 4-8	4-11 5-11	6-3 7-7	7-8 9-3	UNCOATED EPOXY
	$1.3\ \ell_d$	OTHERS	1-1 1-3	1-4 2-0	1-7 2-5	2-2 3-2	2-9 4-2	3-6 5-3	4-5 6-8	5-6 8-2	UNCOATED EPOXY
	CLASS C	TOP [1]	1-11 2-4	2-5 2-11	2-11 3-6	3-10 4-8	5-1 6-2	6-5 7-9	8-1 9-10	10-0 12-1	UNCOATED EPOXY
	$1.7\ \ell_d$	OTHERS	1-5 1-8	1-9 2-7	2-1 3-1	2-9 4-2	3-8 5-5	4-7 6-10	5-10 8-8	7-2 10-8	UNCOATED EPOXY
LESS THAN 6"	CLASS A	TOP [1]	1-5 1-9	1-9 2-2	2-2 2-7	2-10 3-5	3-9 4-6	4-9 5-9	6-0 7-3	7-4 8-11	UNCOATED EPOXY
	$1.0\ \ell_d$	OTHERS	1-0 1-3	1-3 1-11	1-6 2-3	2-1 3-1	2-8 4-0	3-5 5-1	4-3 6-5	5-3 7-10	UNCOATED EPOXY
	CLASS B	TOP [1]	1-10 2-3	2-4 2-10	2-9 3-4	3-8 4-6	4-10 5-10	6-1 7-5	7-9 9-5	9-6 11-7	UNCOATED EPOXY
	$1.3\ \ell_d$	OTHERS	1-4 1-7	1-8 2-6	2-0 3-0	2-8 3-11	3-6 5-2	4-5 6-7	5-7 8-4	6-10 10-2	UNCOATED EPOXY
	CLASS C	TOP [1]	2-5 2-11	3-0 3-8	3-7 4-5	4-10 5-10	6-4 7-8	8-0 9-8	10-2 12-4	12-5 15-1	UNCOATED EPOXY
	$1.7\ \ell_d$	OTHERS	1-9 2-1	2-2 3-3	2-7 3-10	3-5 5-2	4-6 6-9	5-9 8-7	7-3 10-10	8-11 13-4	UNCOATED EPOXY

Table 9.9-1

Tension Lap Splice Length or Development Length - Deformed Bars

LRFD [5.11.2.1, 5.11.5.3.1]

[1] Top Bar – is a horizontal or nearly horizontal bar with 12 inches of fresh concrete cast below it.

CLASS A - [A_s provided/A_s required] ≥ 2; Bars spliced are 75% or less.

CLASS B - [A_s provided/A_s required] < 2; Bars spliced are 50% or less (or) [A_s provided/A_s required] ≥ 2; Bars spliced are greater than 75%.

CLASS C - [A_s provided/A_s required] < 2; Bars spliced are greater than 50%.

(f'_c = 4000 psi; f_y = 60 ksi)											
BAR SPACING	BAR SIZE		4	5	6	7	8	9	10	11	TYPE
6" OR MORE	CLASS A 1.0 ℓ_d	TOP [1]	1-2	1-5	1-9	2-2	2-10	3-6	4-6	5-6	UNCOATED
			1-5	1-9	2-1	2-7	3-5	4-3	5-5	6-8	EPOXY
		OTHERS	1-0	1-0	1-3	1-6	2-0	2-6	3-3	3-11	UNCOATED
			1-0	1-6	1-10	2-3	3-0	3-9	4-10	5-11	EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-6	1-10	2-3	2-9	3-8	4-7	5-10	7-2	UNCOATED
			1-9	2-3	2-8	3-4	4-5	5-7	7-1	8-8	EPOXY
		OTHERS	1-1	1-4	1-7	2-0	2-7	3-3	4-2	5-1	UNCOATED
			1-3	2-0	2-5	3-0	3-11	4-11	6-3	7-8	EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	1-11	2-5	2-11	3-7	4-9	6-0	7-7	9-4	UNCOATED
			2-4	2-11	3-6	4-5	5-9	7-3	9-3	11-4	EPOXY
		OTHERS	1-5	1-9	2-1	2-7	3-5	4-3	5-5	6-8	UNCOATED
			1-8	2-7	3-1	3-10	5-1	6-5	8-2	10-0	EPOXY
LESS THAN 6"	CLASS A 1.0 ℓ_d	TOP [1]	1-5	1-9	2-2	2-8	3-6	4-5	5-7	6-10	UNCOATED
			1-9	2-2	2-7	3-3	4-3	5-4	6-9	8-4	EPOXY
		OTHERS	1-0	1-3	1-6	1-11	2-6	3-2	4-0	4-11	UNCOATED
			1-3	1-11	2-3	2-10	3-9	4-9	6-0	7-4	EPOXY
	CLASS B 1.3 ℓ_d	TOP [1]	1-10	2-4	2-9	3-5	4-6	5-9	7-3	8-11	UNCOATED
			2-3	2-10	3-4	4-2	5-6	6-11	8-10	10-10	EPOXY
		OTHERS	1-4	1-8	2-0	2-6	3-3	4-1	5-2	6-5	UNCOATED
			1-7	2-6	3-0	3-8	4-10	6-2	7-9	9-7	EPOXY
	CLASS C 1.7 ℓ_d	TOP [1]	2-5	3-0	3-7	4-6	5-11	7-6	9-6	11-8	UNCOATED
			2-11	3-8	4-5	5-6	7-2	9-1	11-6	14-2	EPOXY
		OTHERS	1-9	2-2	2-7	3-3	4-3	5-4	6-9	8-4	UNCOATED
			2-1	3-3	3-10	4-10	6-4	8-0	10-2	12-6	EPOXY

Table 9.9-2

Tension Lap Splice Length or Development Length – Deformed Bars

LRFD [5.11.2.1, 5.11.5.3.1]

[1] Top Bar – is a horizontal or nearly horizontal bar with 12 inches of fresh concrete cast below it.

CLASS A – [A_s provided/A_s required] ≥ 2; Bars spliced are 75% or less.

CLASS B – [A_s provided/A_s required] < 2; Bars spliced are 50% or less (or) [A_s provided/A_s required] ≥ 2; Bars spliced are greater than 75%.

CLASS C - [A_s provided/A_s required] < 2; Bars spliced are greater than 50%.

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removed by the roadway contractor. A semi-retaining or full-retaining abutment placed on fill is considered a unique problem by the design engineer, and limits of excavation must be set accordingly. Construction sequence and type of fill material are considered when setting excavation limits. Slopes greater than 1.5 horizontal to 1 vertical are difficult to construct and generally are not specified. It is sometimes advantageous to have the roadway contractor place extra fill that later must be excavated by the bridge contractor, because the overburden aids in compaction and reduces subsequent settlement.

Lateral limits of excavation are not defined in the *Standard Specifications*. The contractor must excavate whatever is necessary within the right-of-way for the placement of the forms.

12.6 Abutment Drainage and Backfill

This section describes abutment design considerations related to drainage and backfill. The abutment drainage and backfill must be designed and detailed properly to prevent undesirable loads from being applied to the abutment.

12.6.1 Abutment Drainage

Abutment drainage is necessary to prevent hydrostatic pressure and frost pressure. Hydrostatic pressure, including both soil and water, can amount to an equivalent fluid unit weight of soil of 85 pcf. Frost action, which can occur in silty backfill, may result in extremely high pressures. On high abutments, these pressures will produce a very large force which could result in structural damage or abutment movement if not accounted for in the design.

To prevent these additional pressures on abutments, it is necessary to drain away whatever water accumulates behind the body and wings. This is accomplished using a pervious granular fill on the inside face of the abutment. Pipe underdrain must be provided to drain the fill located behind the abutment body and wings. For rehabilitation of structures, provide plan details to replace inadequate underdrain systems.

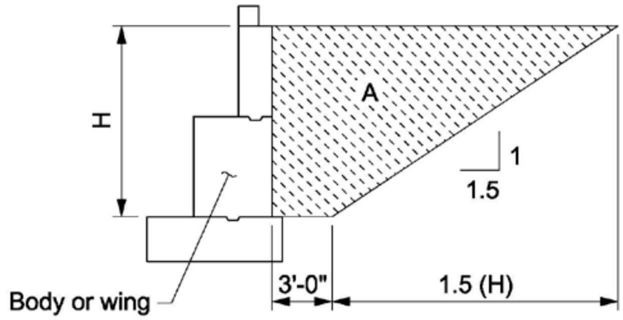
Past experience indicates that sill abutments are not capable of withstanding hydrostatic pressure on their full height without leaking.

Semi-retaining and full-retaining abutments generally will be overstressed or may slide if subject to large hydrostatic or frost pressures unless accounted for in the design. Therefore, “Pipe Underdrain Wrapped 6-inch” is required behind all abutments. This pipe underdrain is used behind the abutment and outside the abutment to drain the water away. It is best to place the pipe underdrain at the bottom of footing elevation as per standards. However, if it is not possible to discharge the water to a lower elevation, the pipe underdrain should be placed higher.

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

12.6.2 Abutment Backfill Material

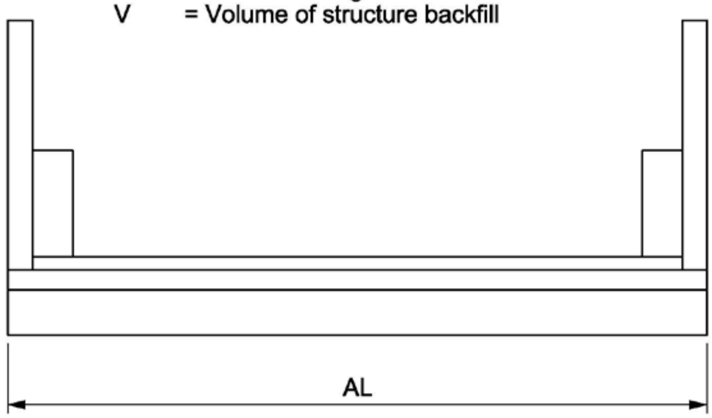
All abutments and wings shall utilize “Backfill Structure” to facilitate drainage. The limits for calculating the material quantity are as shown in Figure 12.6-1 for structures that do not utilize structural approach slabs. For structures that do utilize structural approach slabs, Figure 12.6-2 shows the limits for calculating the material quantities for “Base Aggregate Dense 1¼”. These sketches should not be included on the contract plans.



WING ELEVATION

A = 3.0(H) + 0.5(H)(1.5H) = 3.0(H) + 0.75(H^2)
V = (AL)[3.0(H) + 0.75(H^2)] / 27
V = ___ C.Y. (Bid in C.Y.)

A = Section of structure backfill
AL = Abutment length
V = Volume of structure backfill



ABUTMENT PLAN

STRUCTURE BACKFILL

Note: Use AL and H as given on the plan in feet.
Add 20% shrinkage factor for estimating the total quantity.

Figure 12.6-1
Limits for Calculating Backfill Structure for Structures without Structural Approach Slabs

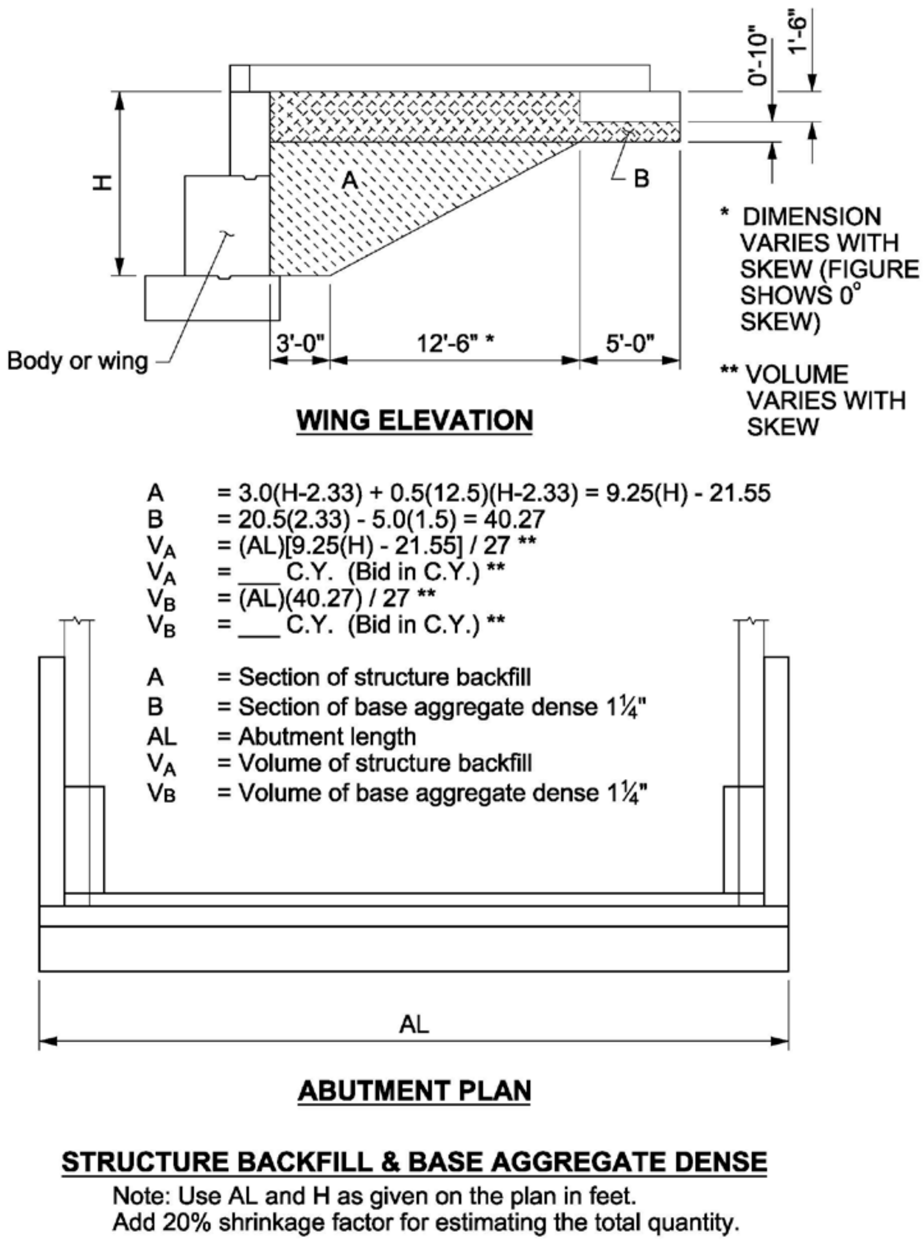


Figure 12.6-2

Limits for Calculating Base Aggregate Dense 1¼" and Backfill Structure with Structural Approach Slabs

12.7 Selection of Standard Abutment Types

From past experience and investigations, the abutment types presented in Figure 12.7-1 are generally most suitable and economical for the given conditions. Although piles are shown for each abutment type, drilled shafts or spread footings may also be utilized depending on the material conditions at the bridge site. The chart in Figure 12.7-1 provides a recommended guide for abutment type selection.

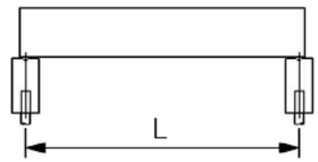
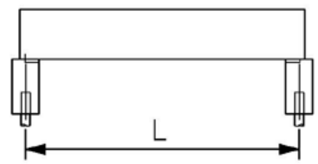
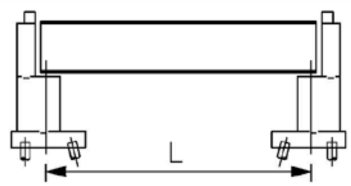
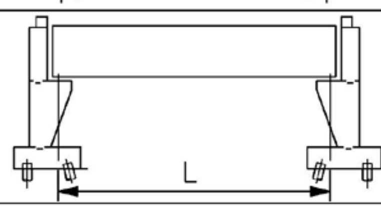
Abutment Arrangements L=Length of continuous superstructure between abutments			Superstructures S=Skew, AL=Abutment Length Concrete Slab Spans	Prestressed Girders	Steel Girders
(1) Type A1 with fixed seat		Type A1 with fixed seat	a. L ≤ 150' S ≤ 30° AL ≤ 50'	a. L ≤ 150' S ≤ 15° AL ≤ 50'	a. L ≤ 150' S ≤ 15° AL ≤ 50'
(2) Type A1 with semi-exp. seat		Type A1 with semi-exp. seat	a. L ≤ 300' S ≤ 30° AL > 50'	a. L ≤ 300' S ≤ 40°	a. L ≤ 150' S ≤ 40°
(3) Type A3 with fixed bearing		Type A3 with exp. bearing	Not used	Single span and (S > 40°)	Single span and (L > 150' or S > 40°)
(4) Type A3 with exp. bearing		Type A3 with exp. bearing	b. L >300' and S ≤ 30° with rigid piers	L>300' or (S > 40° and multi-span)	Multi-span and (L>150' or S > 40°) with rigid piers
(5) Type A4 with exp. bearing		Type A4 with exp. bearing	Not used	c. Based on geometry and economics Girder D < 60"	d. Based on geometry and economics Girder D < 60"
ABUTMENT TYPES					

Figure 12.7-1
Recommended Guide for Abutment Type Selection

Footnotes to Figure 12.7-1:

a. Type A1 fixed abutments are not used when wing piles are required. The semi-expansion seat is used to accommodate superstructure movements and to minimize cracking between the wings and body wall. See Standards for Abutment Type A1 (Integral Abutment) and Abutment Type A1 for additional guidance.

b. Consider the flexibility of the piers when choosing this abutment type. Only one expansion bearing is needed if the structure is capable of expanding easily in one direction. With rigid piers, symmetry is important in order to experience equal expansion movements and to minimize the forces on the substructure units.

c. For two-span prestressed girder bridges, the sill abutment is more economical than a semi-retaining abutment if the maximum girder length is not exceeded. It also is usually more economical if the next girder size is required.

d. For two-span steel structures with long spans, the semi-retaining abutments may be more economical than sill abutments due to the shorter bridge lengths if a deeper girder is required.

12.8 Abutment Design Loads and Other Parameters

This section provides a brief description of the application of abutment design loads, a summary of load modifiers, load factors and other design parameters used for abutment and wing wall design, and a summary of WisDOT abutment design policy items.

12.8.1 Application of Abutment Design Loads

An abutment is subjected to both horizontal and vertical loads from the superstructure. The number and spacing of the superstructure girders determine the number and location of the concentrated reactions that are resisted by the abutment. The abutment also resists loads from the backfill material and any water that may be present.

Although the vertical and horizontal reactions from the superstructure represent concentrated loads, they are commonly assumed to be distributed over the entire length of the abutment wall or stem that support the reactions. That is, the sum of the reactions, either horizontal or vertical, is divided by the length of the wall to obtain a load per unit length to be used in both the stability analysis and the structural design. This procedure is sufficient for most design purposes.

The first step in computing abutment design loads is to compute the dead load reactions for each girder or beam. To illustrate this, consider a 60-foot simple span structure with a roadway width of 44 feet, consisting of steel beams spaced at 9 feet and carrying an HL-93 live loading.

The dead load forces, DC and DW, acting on the abutments shall include reactions from the superstructure. DC dead loads include structural components and nonstructural attachments, and DW dead loads include wearing surfaces and utilities. If the total DC dead load is 1.10 kips per foot of girder and the total DW dead load is 0.18 kips per foot of girder, then the dead load reaction per girder is computed as follows:

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

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

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

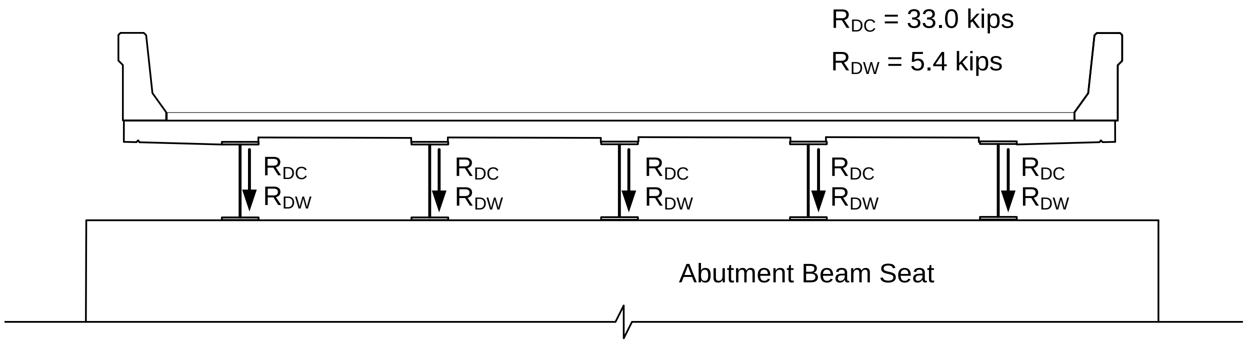


Figure 12.8-1
Dead Load on Abutment Beam Seat

The next step is to compute the live load applied to the abutment. To compute live load reactions to bearings, live load distribution factors must be used to compute the maximum live load reaction experienced by each individual girder. However, to compute live loading on abutments, the maximum number of design lanes are applied to the abutment to obtain the live load per foot of length along the abutment. Live load distribution factors are not used for abutment design, because it is too conservative to apply the maximum live load reaction for each individual girder; each individual girder will generally not experience its maximum live load reaction simultaneously because each one is based on a different configuration of design lane locations.

To illustrate the computation of live loads for abutment design, consider the same 60-foot simple span bridge described previously. Since the roadway width is 44 feet, the maximum number of design lanes is three (44 / 12 = 3.67 ≈ 3 lanes). The backwall live load is computed by placing the three design truck axles along the abutment and calculating the load on a per foot basis. The dynamic load allowance and multiple presence factor shall be included. The load is applied to the entire length of the abutment backwall and is assumed to act at the front top corner (bridge side) of the backwall. This load is not applied, however, when designing the abutment wall (stem) or footing. Assuming an abutment length of 48 feet and a backwall width of 2.0 feet, the backwall live load is computed as follows:

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

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

It should be noted that dynamic load allowance is applied to the truck live load only and not to the lane live load. This live load configuration on the abutment backwall is illustrated in Figure 12.8-2.

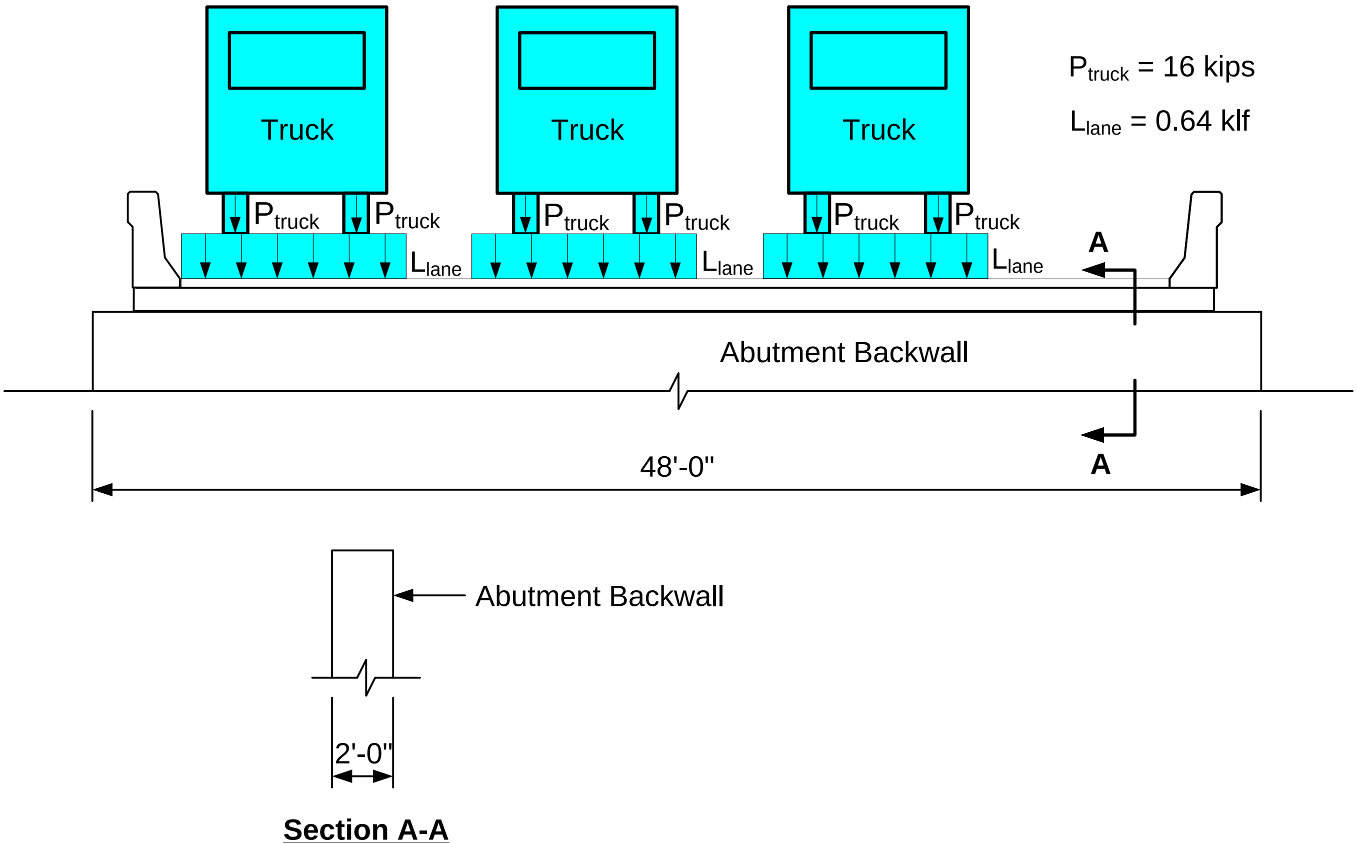


Figure 12.8-2
Live Load on Abutment Backwall

To compute the live loads applied to the abutment beam seat, the live load reactions should be obtained for one lane loaded using girder design software. For this example, for one design lane, the maximum truck live load reaction is 60.8 kips and the maximum lane live load reaction is 19.2 kips. In addition, assume that the abutment is relatively high; the load can therefore be distributed equally over the full length of the abutment. For wall (stem) design, the controlling maximum live loads applied at the beam seat are computed as follows, using three design lanes and using both dynamic load allowance and the multiple presence factor:

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

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

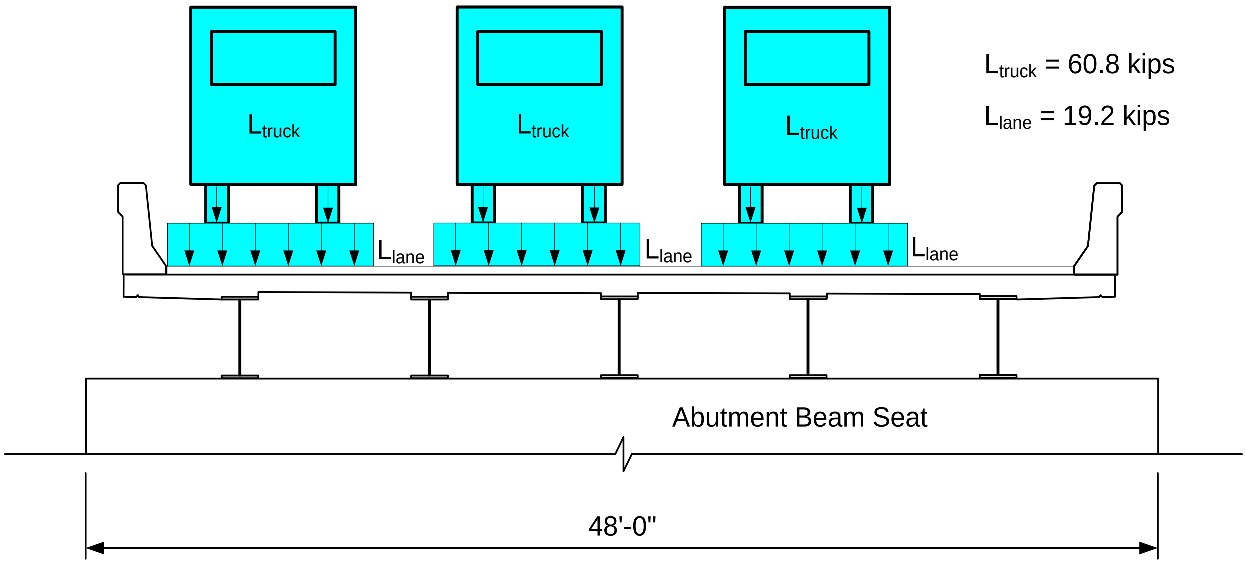


Figure 12.8-3
Live Load on Abutment Beam Seat

For a continuous bridge, the minimum live load applied to the abutment beam seat can be obtained based on the minimum (negative) live load reactions taken from girder design software output.

For footing design, the dynamic load allowance is not included. Therefore, the controlling maximum live loads applied at the beam seat are computed as follows:

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

12.8.2 Load Modifiers and Load Factors

Table 12.8-1 presents the load modifiers used for abutment and wing wall design.

Description	Load Modifier
Ductility	1.00
Redundancy	1.00
Operational classification	1.00

Table 12.8-1
Load Modifiers Used in Abutment Design

Table 12.8-2 presents load factors used for abutment and wing wall design. Load factors presented in this table are based on the Strength I and Service I limit states. The load factor for substructure wind loads is 0.00, because wind loads in the transverse direction may be ignored in abutment and wing wall design.

Direction of Load	Specific Loading	Load Factor		
		Strength I		Service I
		Max.	Min.	
Load factors for vertical loads	Superstructure DC dead load	1.25	0.90	1.00
	Superstructure DW dead load	1.50	0.65	1.00
	Superstructure live load	1.75	1.75	1.00
	Approach slab dead load	1.25	0.90	1.00
	Approach slab live load	1.75	1.75	1.00
	Wheel loads located directly on the abutment backwall	1.75	1.75	1.00
	Earth surcharge	1.50	0.75	1.00
	Earth pressure	1.35	1.00	1.00
	Water load	1.00	1.00	1.00
	Live load surcharge	1.75	1.75	1.00
Load factors for horizontal loads	Substructure wind load	0.00	0.00	0.00
	Superstructure wind load	0.00	0.00	0.30
	Vehicular braking force from live load	1.75	1.75	1.00
	Temperature and shrinkage	1.20	0.50	1.00
	Earth pressure (active)	1.50	0.90	1.00
	Earth surcharge	1.50	0.75	1.00
	Live load surcharge	1.75	1.75	1.00

Table 12.8-2

Load Factors Used in Abutment Design

12.8.3 Live Load Surcharge

The equivalent heights of soil for vehicular loading on abutments perpendicular to traffic are as presented in **LRFD [Table 3.11.6.4-1]** and in Table 12.8-3. Values are presented for various abutment heights. The abutment height, as used in Table 12.8-3, is taken as the distance between the top surface of the backfill at the back face of the abutment and the bottom of the footing along the pressure surface being considered. Linear interpolation should be used for intermediate abutment heights. The load factors for both vertical and horizontal components of live load surcharge are as specified in **LRFD [Table 3.4.1-1]** and in Table 12.8-2.

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

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

WisDOT policy item:

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

For abutments with concrete approach slabs, one half of the equivalent height of soil shall be used to calculate the horizontal load on the abutment. The design lane load shall be applied to the concrete approach slab and this reaction shall be applied as a vertical 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. A value of 1.2 klf shall be used. Do not add live load from the approach slab to the abutment.

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

12.8.6 Horizontal Pile Resistance

The following procedure shall be used to verify the horizontal resistance of the piles for A3 and A4 abutments.

Given information:

Horizontal Loads	Unfactored (klf)		Load Factor		Factored Load (klf)
Earth Pressure	5.5	x	1.50	=	8.25
Live Load Surcharge	1.0	x	1.75	=	1.75
Temp. Load from Bearings	0.6	x	0.50	=	0.30
			Total, Hu	=	10.3

Back row pile spacing =	8.0 feet
Front row pile spacing =	5.75 feet
Ultimate Vertical Resistance, 12 3/4” CIP, Pr =	210 kips per pile

Factored Vertical Load on Front Row Pile*	160 kips per pile
Ultimate Horizontal Resistance of back row pile (from Geotech Report), Hr =	14 kips per pile
Ultimate Horizontal Resistance of front row pile (from Geotech Report), Hr =	11 kips per pile

* When calculating the horizontal component of the battered pile, use the actual factored load on the pile resulting from the loading conditions where the horizontal loads are maximized and the vertical loads are minimized.

Calculate horizontal component of the battered pile. The standard pile batter is 1:4.

$$Hr_{battered} = 160\left(\frac{1}{\sqrt{1^2 + 4^2}}\right)$$

$Hr_{battered}$ = 38.8 kips per pile

Calculate ultimate resistance provided by the pile configuration:

$$Hr = \left(\frac{14}{8.0}\right) + \left(\frac{11}{5.75}\right) + \left(\frac{38.8}{5.75}\right)$$

Hr = 10.4 klf

Hr > Hu = 10.3 klf OK

12.9 Abutment Body Details

There are many different body sections that are utilized for each of the different abutment types. When designing these sections, it is inadvisable to use small and highly reinforced sections. As a general principle, it is better to use a lot of concrete and less reinforcing steel, thus making parts relatively massive and stiff. Adequate horizontal reinforcement and vertical contraction joints are essential to prevent cracking, especially when wing walls are poured monolithically with the abutment body.

The bottom of abutment bodies are normally constructed on a horizontal surface. However, abutments constructed on a horizontal surface may require one end of the body to be much higher than the opposite end due to the vertical geometry of the bridge. This sometimes requires an extremely long and high wing wall. For these extreme cases, the bottom of the abutment body can be stepped.

The berm in front of the body is held level even though the body is stepped. A minimum distance of 2.5 feet between the top of berm and the top of beam seat is allowed. Minimum ground cover as shown in the Standard Detail for Abutments must be maintained.

Stepping the bottom of the body may result in a longer bridge. This is usually more costly than holding the body level and using larger wings and beam seats. Stepped abutments are also more difficult to build. Engineering judgment must be exercised when determining if the bottom of the abutment should be level or stepped. Generally, if a standard wing wall design cannot be used, the bottom of the abutment body should be stepped.

12.9.1 Construction Joints

In a U-shaped abutment with no joint between the wings and the body, traffic tends to compact the fill against the three sides of the abutment. When the temperature drops, the abutment body concrete cannot shrink without tending to squeeze the warmer fill inside. The resistance of the fill usually exceeds the tensile or shearing strength of the body or wing, and cracks result.

If contraction joints are not provided in long abutment bodies, nature usually creates them. To prevent uncontrolled cracking in the body or cracking at the body-wing joint, body pours are limited to a maximum of 50 feet. Expansion joints are required at a maximum of 90 feet, as specified in **LRFD [11.6.1.6]**.

WisDOT exception to AASHTO:

LRFD [11.6.1.6] specifies that contraction joints shall be provided at intervals not exceeding 30 feet for conventional retaining walls and abutments. However, WisDOT has not experienced significant problems with 50 feet and uses a maximum interval of 50 feet.

Shear keys are provided in construction joints to allow the center pour to maintain the beneficial stabilizing effects from the wings. The shear keys enable the end pours, with their counterfort action due to the attached wing, to provide additional stability to the center pour. Reinforcing steel should be extended through the joint.

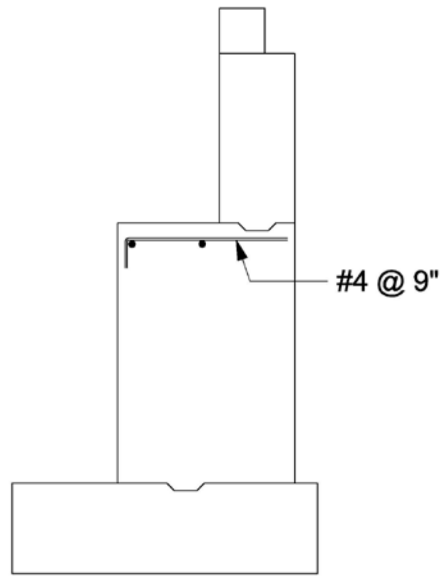
In general, body construction joints are keyed to hold the parts in line. Water barriers are used to prevent leakage and staining. Steel girder superstructures generally permit a small movement at construction joints without cracking the concrete slab. In the case of concrete slab or prestressed concrete girder construction, a crack will frequently develop in the deck above the abutment construction joint. The designer should consider this when locating the construction joint.

12.9.2 Beam Seats

Because of the bridge deck cross-slopes or skewed abutments, it is necessary to provide beam seats of different elevations on the abutment. The tops of these beam seats are poured to precise elevations and are made level except when elastomeric bearing pads are used and grades are equal to or exceed 1%. For this case, the beam seat should be parallel to the bottom of girder or slab. Shrinkage and practical difficulties in obtaining good workmanship make it difficult to obtain the exact beam seat elevation.

When detailing abutments, the differences in elevations between adjacent beam seats are provided by sloping the top of the abutment between level beam seats. For steel girders, the calculation of beam seat elevations and use of shim plates is described on the Standard *Plate Girder Details* in Chapter 24.

The tops of bearing seats are usually subjected to very large localized pressures under the bearings. Additional reinforcement directly under the bearings is sometimes necessary to prevent the formation of visible cracks or possible spalling of concrete. This additional reinforcement is required for seats that are stepped 4” or more when the standard body reinforcement is not sufficient to prevent the possibility of this cracking or spalling. A common detail includes a grid of #4 reinforcing bars bent down into the abutment body, as shown in Figure 12.9-1.



BODY SECTION

Showing Grid Detail In Bearing Seat

Figure 12.9-1

Reinforcing Grid Detail in Bearing Seat

12.10 Timber Abutments

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.

12.11 Bridge Approach Design and Construction Practices

While most bridge approaches are reasonably smooth and require a minimum amount of maintenance, there are also rough bridge approaches with maintenance requirements. The bridge designer should be aware of design and construction practices that minimize bridge approach maintenance issues. Soils, design, construction and maintenance engineers must work together and are jointly responsible for efforts to eliminate rough bridge approaches.

An investigation of the foundation site is important for bridge design and construction. The soils engineer, using tentative grades and foundation site information, can provide advice on the depth of material to be removed, special embankment foundation drainage, surcharge heights, waiting periods, construction rates and the amount of post-construction settlement that can be anticipated. Some typical bridge approach problems include the following:

- Settlement of pavement at end of approach slab
- Uplift of approach slab at abutment caused from swelling soils or freezing
- Backfill settlement under flexible pavement
- Approach slab not adequately supported at the abutments
- Erosion due to water infiltration

Most bridge approach problems can be minimized during design and construction by considering the following:

- Embankment height, material and construction methods
- Subgrade, subbase and base material
- Drainage-runoff from bridge, surface drains and drainage channels
- Special approach slabs allowing for pavement expansion

Post-construction consolidation of material within the embankment foundation is the primary contributor to rough bridge approaches. Soils which consist predominantly of sands and gravels are least susceptible to consolidation and settlement. Soils with large amounts of shales, silts and plastic clays are highly susceptible to consolidation.

The following construction measures can be used to stabilize foundation materials:

- Consolidate the natural material. Allow sufficient time for consolidation under the load of the embankment. When site investigations indicate an excessive length of time is required, other courses of corrective action are available. Use of a surcharge fill is effective where the compressive stratum is relatively thin and sufficient time is available for consolidation.

- Remove the material either completely or partially. This procedure is practical if the foundation depth is less than 15 feet and above the water table.

- Use lightweight embankment materials. Lightweight materials (fly ash, expanded shale and cinders) have been used with apparent success for abutment embankment construction to lessen the load on the foundation materials.

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

- Use of select materials

- Placement of relatively thin 4- to 6-inch layers

- Strict control of moisture and density

- Proper compaction

- Installation of moisture barriers

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

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

WisDOT policy item:

Structural approach slabs shall be used on all Interstate Highway bridges and U.S.H. bridges. Other locations can be considered with the approval of the Chief Structural Design Engineer.

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

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In Figure 13.7-3, the strut area is influenced by the stirrup spacing, s, as well as the diameter of the longitudinal tension steel, d_{ba}. In Figure 13.7-4, the strut area is influenced by the bearing dimensions, L_b, in both directions, as well as the location of the center of gravity of the longitudinal tension steel, $0.5h_a$. In Figure 13.7-5, the strut area is influenced by the bearing dimensions, L_b, in both directions, as well as the height of the compression strut, h_s. The value of h_s shall be taken as equal to "a" as shown in Figure 13.7-2. The strut area in each of the three previous figures depends upon the angle of the strut with respect to the horizontal, θ_s.

If the initial strut width is inadequate to develop the required resistance, the engineer should increase the bearing block size.

13.7.6 Check the Tension Tie Anchorage

Tension ties shall be anchored to the nodal zones by either specified embedment length or hooks so that the tension force may be transferred to the nodal zone. As specified in **LRFD [5.6.3.4]**, the tie reinforcement shall be fully developed at the inner face of the nodal zone. In Figure 13.7-4, this location is given by the edge of the bearing where θ_s is shown.

13.7.7 Provide Crack Control Reinforcement

Pier caps designed using the strut-and-tie method shall contain an orthogonal grid of reinforcing bars near each face in accordance with **LRFD [5.6.3.6]**. This reinforcement will control crack widths and ensure a minimum ductility. The ratio of reinforcement area to gross concrete area shall not be less than 0.003 in both directions. Maximum bar spacing shall not exceed 12". The crack control steel, when located within the tension tie, may be considered as part of the tension tie reinforcement.

13.8 General Pier Cap Information

The minimum cap dimension to be used is 3' deep by 2'-6" wide, with the exception that a 2'-6" deep section may be used for caps under slab structures. If a larger cap is needed, use 6" increments to increase the size. The multi-column cap width shall be a minimum of 1 1/2" wider than the column on each side to facilitate construction forming. The pier cap length shall extend a minimum of 2' transversely beyond the centerline of bearing and centerline of girder intersection.

On continuous slab structures, the moment and shear forces are proportional between the transverse slab section and the cap by the ratio of their moments of inertia. The effective slab width assumed for the transverse beam is the minimum of 1/2 the center-to-center column spacing or 8.0'.

$$M_{cap} = M_{total} \frac{I_{cap}}{I_{cap} + I_{slab}}$$

Where:

M_{cap}	=	Cap moment (kip-ft)
M_{total}	=	Total moment (kip-ft)
I_{cap}	=	Moment of inertia of pier cap (in^4)
I_{slab}	=	Moment of inertia of slab (in^4)

The concrete slab is to extend beyond the edge of pier cap as shown on Standards for Continuous Haunched Slab and for Continuous Flat Slab. If the cap is rounded, measure from a line tangent to the pier cap end and parallel to the edge of the deck.

Reinforcement bars are placed straight in the pier cap. Determine bar cutoff points on wide caps. If the pier cap is cantilevered over exterior columns, the top negative bar steel may be bent down at the ends to ensure development of this primary reinforcement.

Do not place shear stirrups closer than 4" on centers. Generally only double stirrups are used, but triple stirrups may be used to increase the spacing. If these methods do not work, increase the cap size. Stirrups are generally not placed over the columns. The first stirrup is placed one-half of the stirrup spacing from the edge of the column into the span.

The cap-to-column connection is made by extending the column reinforcement straight into the cap the necessary development length. Stirrup details and bar details at the end of the cap are shown on Standard for Multi-Columned Pier.

Crack control, as defined in **LRFD [5.7.3.4]** shall be considered for pier caps. Class 2 exposure condition exposure factors shall only be used when concern regarding corrosion (i.e., pier caps located below expansion joints, pier caps subject to intermittent moisture above waterways, etc.) or significant aesthetic appearance of the pier cap is present.

13.9 Column / Shaft Design

See 13.4.10 for minimum shaft design requirements regarding the Extreme Event II collision load of **LRFD [3.6.5]**.

Use an accepted analysis procedure to determine the axial load as well as the longitudinal and transverse moments acting on the column. These forces are generally largest at the top and bottom of the column. Apply the load factors for each applicable limit state. The load factors should correspond to the gross moment of inertia. Load factors vary for the gross moment of inertia versus the cracked moment as defined in **LRFD [3.4.1]** for γ_{TU}, γ_{CR}, γ_{SH}. Choose the controlling load combinations for the column design.

Columns that are part of a pier frame have transverse moments induced by frame action from vertical loads, wind loads on the superstructure and substructure, wind loads on live load, thermal forces and centrifugal forces. If applicable, the load combination for Extreme Event II must be considered. Longitudinal moments are produced by the above forces, as well as the braking force. These forces are resolved through the skew angle of the pier to act transversely and longitudinally to the pier frame. Longitudinal forces are divided equally among the columns.

Wisconsin uses tied columns following the procedures of **LRFD [5.7.4]**. The minimum allowable column size is 2'-6" in diameter. The minimum steel bar area is as specified in **LRFD [5.7.4.2]**. For piers not requiring a certain percentage of reinforcement as per 13.4.10 to satisfy **LRFD [3.6.5]** for vehicular collision load, a reduced effective area of reinforcement may be used when the cross-section is larger than that required to resist the applied loading.

The computed column moments are to consider moment magnification factors for slenderness effects as specified in **LRFD [5.7.4.3]**. Values for the effective length factor, K, are as follows:

- 1.2 for longitudinal moments with a fixed seat supporting prestressed concrete girders
- 2.1 for longitudinal moments with a fixed seat supporting steel girders and all expansion bearings
- 1.0 for all transverse moments

The computed moments are multiplied by the moment magnification factors, if applicable, and the column is designed for the combined effects of axial load and bending. According to **LRFD [5.7.4.1]** all force effects, including magnified moments, shall be transferred to adjacent components. The design resistance under combined axial load and bending is based on stress-strain compatibility. A computer program is recommended for determining the column's resistance to the limit state loads.

As a minimum, the column shall provide the steel shown on the Pier Standards.

On large river crossings, it may be necessary to protect the piers from damage. Dolphins may be provided.

The column-to-cap connection is designed as a rigid joint considering axial and bending stresses. Column steel is run through the joint into the cap to develop the compressive stresses or the tensile steel stresses at the joint.

In general, the column-to-footing connection is also designed as a rigid joint. The bar steel from the column is generally terminated at the top of the footing. Dowel bars placed in the footing are used to transfer the steel stress between the footing and the column.

Crack control, as defined in **LRFD [5.7.3.4]** shall be considered for pier columns. All pier columns shall be designed using a Class 2 exposure condition exposure factor.

13.9.1 Tapered Columns of Concrete and Timber

Design tapered columns of concrete and timber using the existing column formulas, taking the cross-sectional area at the small end. However, d, as used in L/d, is taken as follows:

1. For round columns or rectangular columns tapered in both directions, use:

$$d = d_B$$

2. For rectangular columns tapered in the plane of bending only, use:

$$d = (d_A)2(d_B)8$$

3. For rectangular columns tapered perpendicular to the plane of bending, use:

$$d = (d_A)7(d_B)3$$

Where:

d_A = Dimension at the small end

d_B = Dimension at the large end

13.10 Pile Bent and Pile Encased Pier Analysis

WisDOT policy item:

Only the Strength I limit state need be utilized for determining the pile configuration required for open pile bents and pile encased piers. Longitudinal forces are not considered due to fixed or semi-expansion abutments being required for these pier types.

The distribution of dead load to the pile bents and pile encased piers is in accordance with 17.2.9. Live load is distributed according to 17.2.10.

WisDOT policy item:

Dynamic load allowance, IM, is included for determining the pile loads in pile bents, but not for piling in pile encased piers.

The pile force in the outermost, controlling pile is equal to:

$$P_n = \frac{F}{n} + \frac{M}{S}$$

Where:

F = Total factored vertical load (kips)

n = Number of piles

M = Total factored moment about pile group centroid (kip-ft)

S = Section modulus of pile group (ft^3), equal to:

$$\left(\frac{\sum d^2}{c}\right)$$

In which:

d = Distance of pile from pile group centroid

c = Distance from outermost pile to pile group centroid

See Standard for Pile Bent for details. See Standard for Pile Encased Pier for details.

13.11 Footing Design

13.11.1 General Footing Considerations

There are typical concepts to consider when designing and detailing both spread footings and pile footings.

For multi-columned piers:

- Each footing for a given pier should be the same dimension along the length of the bridge.
- Each footing for a given pier should be the same thickness.
- Footings within a given pier need not be the same width.
- Footings within a given pier may have variable reinforcement.
- Footings within a given pier may have a different number of piles. Exterior footings should only have fewer piles than an interior footing if the bridge is unlikely to be widened in the future. An appropriate cap span layout will usually lend itself to similar footing/pile configurations.
- Heavier piles, especially if primarily end bearing piles, can be more economical.

For hammerhead piers:

- Make as many seals the same size as reasonable to facilitate cofferdam re-use.
- Seal thickness can vary from pier to pier.
- Footing dimensions, reinforcement and pile configuration can vary from pier to pier.
- Heavier piles, especially if primarily end bearing piles, can be more economical.

WisDOT exception to AASHTO:

Crack control, as defined in **LRFD [5.7.3.4]** shall not be considered for pier isolated spread footings, isolated pile footings and continuous footings.

Shrinkage and temperature reinforcement, as defined in **LRFD [5.10.8]** shall not be considered for side faces of any buried footings.

13.11.2 Isolated Spread Footings

WisDOT policy item:

Spread footings are designed using LRFD strength limit state loads and resistance for moment and shear as specified in **LRFD [5.13.3]**. The foundation bearing capacity, used to dimension the footing's length and width, shall be determined using 4th Edition of the AASHTO *LRFD Bridge Design Specifications for Highway Bridges*.

The spread footing is proportioned so that the foundation bearing capacity is not exceeded. The following steps are used to design spread footings:

1. Minimum depth of spread footings is 2'. Depth is generally determined from shear strength requirements. Shear reinforcement is not used.

2. A maximum of 25% of the footing area is allowed to act in uplift (or nonbearing). When part of a footing is in uplift, its section properties for analysis are based only on the portion of the footing that is in compression (or bearing). When determining the percent of a footing in uplift, use the Service Load Design method.

3. Soil weight on footings is based only on the soil directly above the footing.

4. The minimum depth for frost protection from top of ground to bottom of footing is 4'.

5. Spread footings on seals are designed by either of the following methods:

    a. The footing is proportioned so the pressure between the bottom of the footing and the top of the seal does not exceed the foundation bearing capacity and not more than 25% of the footing area is in uplift.

    b. The seal is proportioned so that pressure at the bottom of the seal does not exceed the foundation bearing capacity and the area in uplift between the footing and the seal does not exceed 25%.

6. The spread footing's reinforcing steel is determined from the flexural requirements of **LRFD [5.7.3]**. The design moment is determined from the volume of the pressure diagram under the footing which acts outside of the section being considered. The weight of the footing and the soil above the footing is used to reduce the bending moment.

7. The negative moment which results if a portion of the footing area is in uplift is ignored. No negative reinforcing steel is used in spread footings.

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

    a. Two-way action

The volume of the pressure diagram on the footing area outside the critical perimeter lines (placed at a distance d/2 from the face of the column, where d equals the effective footing depth) determines the shear force. The shear resistance is influenced by the concrete strength, the footing depth and the critical perimeter length. The critical perimeter length is 2 (L + d + W + d) for rectangular columns and π (2R + d) for round columns, where R is the column radius and d is the effective footing depth. The critical perimeter location for spread footings with rectangular columns is illustrated in Figure 13.11-1.

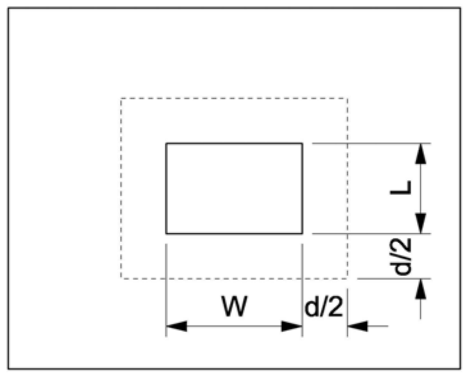

Figure 13.11-1
Critical Perimeter Location for Spread Footings

b. One-way action

The volume of the pressure diagram on the area enclosed by the footing edges and a line placed at a distance "d" from the face of the column determines the shear force. The shear resistance is influenced by the concrete strength, the footing depth and the entire footing width or length. The shear location for one-way action is illustrated in Figure 13.11-2.

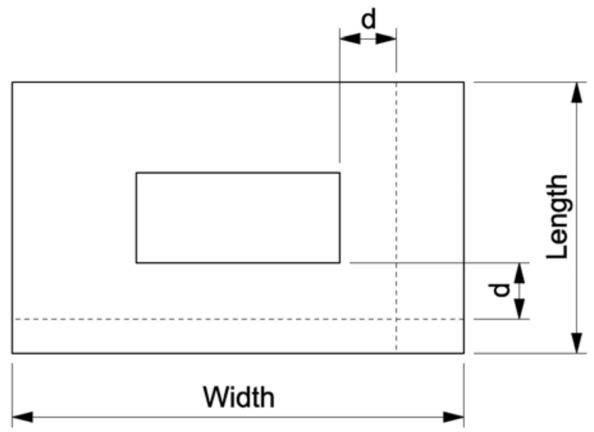

Figure 13.11-2
Shear Location for One-Way Action

The footing weight and the soil above the areas are used to reduce the shear force.

9. The bottom layer of reinforcing steel is placed 3” clear from the bottom of the footing.

10. If adjacent edges of isolated footings are closer than 4’-6”, a continuous footing shall be used.

13.11.3 Isolated Pile Footings

WisDOT policy item:

Pile footings are designed using LRFD strength limit state loads and resistance for moment and shear as specified in **LRFD [5.13.3]**. The pile design shall use LRFD strength limit state loads to compare to the factored axial compression resistance specified in Table 11.3-5.

The nominal geotechnical pile resistance shall be provided in the Site Investigation Report. The engineer shall then apply the appropriate resistance factor from Table 11.3.1 to the nominal resistance to determine the factored pile resistance. The footing is proportioned so that when it is loaded with the strength limit state loads, the factored pile resistance is not exceeded.

The following steps are used to design pile-supported footings:

1. The minimum depth of pile footing is 2’-6”. The minimum pile embedment is 6”. See 13.2.2 for additional information about pile footings used for pile bents.

2. Pile footings in uplift are usually designed by method (a) stated below. However, method (b) may be used if there is a substantial cost reduction.

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

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

3. Same as spread footing.

4. Same as spread footing.

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

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

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

8. The pile footing reinforcing steel is determined from the flexural requirements of **LRFD [5.7.3]**. The design moment and shear are determined from the force of the piles which act outside of the section being considered. The weight of the footing and the soil above the footing are used to reduce the magnitude of the bending moment and shear force.

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

   a. Two-way action

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

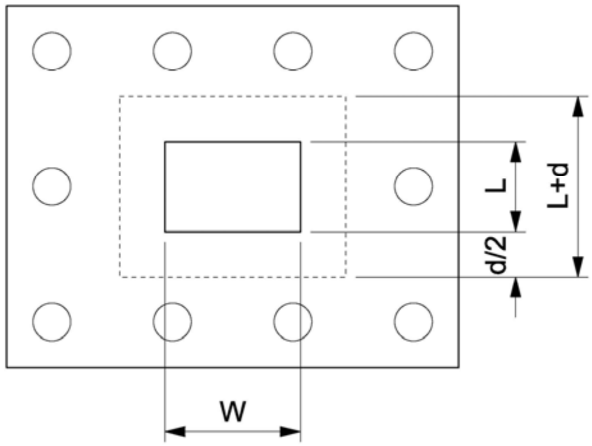

Figure 13.11-3
Critical Perimeter Location for Pile Footings

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

b. One-way action

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

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

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

13.11.4 Continuous Footings

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

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

To prevent unequal settlement, proportion the continuous footing so that soil pressures or pile loads are constant for Service I load combination. The footing should be kept relatively stiff between columns to prevent upward footing deflections which cause excessive soil or pile loads under the columns.

13.11.5 Seals and Cofferdams

A seal is a mat of unreinforced concrete poured under water inside the cofferdam or sheet piling. The seal is designed to withstand the hydrostatic pressure on its bottom when the water above it is removed. Dewatering the cofferdam allows cutting of piles, placement of reinforcing steel and pouring of the footing in a dry environment.

Seals are required for all piers founded on spread or pile footings that are too far below normal water to pour the footing in the dry, which ensures properly consolidated concrete. This criteria is very important for tall piers.

The hydrostatic pressure under the seal is resisted by the seal weight, the friction between the seal perimeter and the cofferdam walls, and friction between seal and piles for pile footings. The friction values used for the seal design are considered using the service limit state. To compute the capacity of piles in uplift, refer to Chapter 11. Values for bond on piles and sheet piling are presented in Table 13.11-1.

Application	Value of Bond
Bond on Piles	10 psi
Bond on Sheet Piling	2 psi applied to [(Seal Depth - 2’) x Seal Perimeter]

Table 13.11-1
Bond on Piles and Sheet Piling

Lateral forces from stream flow pressure are resisted by the penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. The cofferdam design shall be the responsibility of the contractor. When seals for spread footings are founded on rock, the weight of the seal is used to counterbalance the lateral stream flow pressure.

The downstream side of the cofferdam should be keyed into rock deep enough or backfilled enough to resist the lateral stream flow pressure. To provide a factor of safety, the cofferdam weight (sheet piling and bracing) is ignored in the analysis. The design stream flow velocity is based on the flow at the site at the time of construction but need not exceed 75% of the 100-year velocity. The force is calculated as per 13.4.6. If a stream velocity higher than design occurs, the contractor may add water to the cofferdam to improve stability. The extra water weight above the seal adds additional overturning resistance and also increases lateral resistance from additional friction forces at the bottom of the seal.

A rule of thumb for seal thickness is 0.40H for spread footings and 0.25H for pile footings, where H is the water depth from bottom of seal to top of water. The 2-year high water elevation, if available, should be used as the water elevation. The assumed water elevation used to determine the seal thickness should be noted on the plans. The minimum seal size is 1'-6" larger than the footing size on all sides. See Standard for Hammerhead Pier for additional guidance regarding the sizing of the seal.

Example: Determine the seal thickness for a 9' x 12' footing with 12-12" diameter piles. Uplift capacity of one pile equals 10 kips. The water depth to the top of seal is 16'.

Assume 12' x 15' x 3' seal.

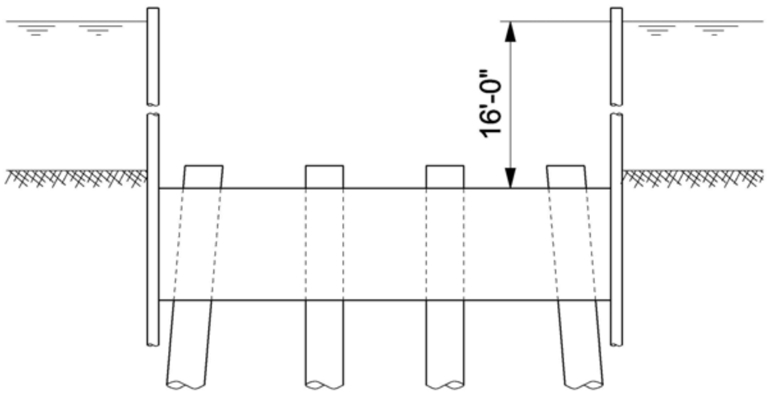

Figure 13.11-4
Seal Inside a Cofferdam

Uplift force of water	12 x 15 x 19 x .0624	=	214 kips up
Weight of seal course	12 x 15 x 3 x .15	=	81 kips down
Friction of sheet piling	2 x (12+15) x 1 x 144 x .002	=	2 x (12+15) x 1 x 144 x .002
		=	16 kips down
Friction on 12-inch diameter pile	p x 12 x 36 x .010	=	13.6 kips
Maximum uplift/pile		=	10 kips
Total available force from piles	12 x 10	=	120 kips down
Summation of downward forces		=	217 kips
	217 > 214 OK		

USE 3'-0" THICK SEAL

A cofferdam is required when the depth of water from the bottom of the seal to the top of the water is 15 feet or greater. Cofferdams may be required when this depth is less than 15 feet for certain soil conditions and stream flows. The designer should consult with geotechnical personnel for these cases. A bid item for Cofferdam should be shown if the possibility of one exists.

13.12 Quantities

Consider the "Upper Limits for Excavation" for piers at such a time when the quantity is a minimum. This is either at the existing ground line or the finished graded section. Indicate in the general notes which value is used.

Granular backfill is not used at piers except under special conditions.

Compute the concrete quantities for the footings, columns and cap, and show values for each of them on the final plans.

13.13 Appendix A – Pier Details

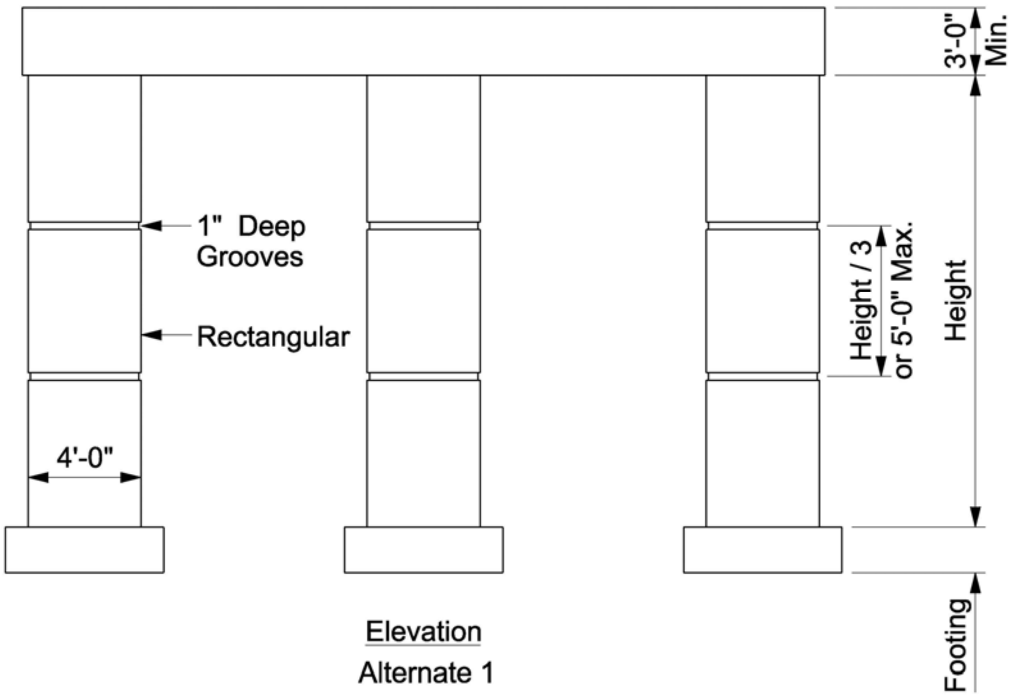

Figure 13.13-1
Multi-Columned Pier on Isolated Spread Footings

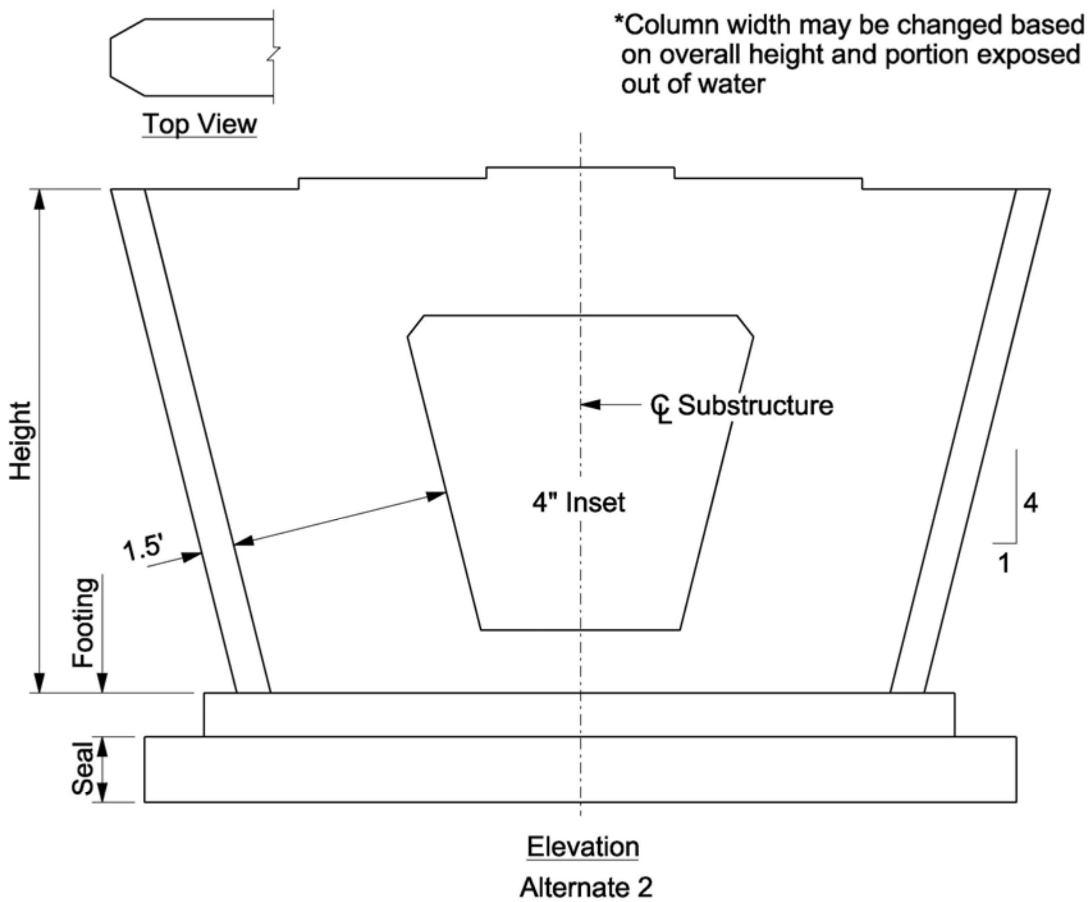

Figure 13.13-2
Wall-Type Pier on Spread Footing and Seal

13.14 Appendix B – Pile Encased Pier Construction

The following procedures shall be employed in the construction of pile encased piers which are constructed without cofferdams. The contractor shall have the option of driving piling prior to excavation. Excavation shall proceed in the following manner.

The area occupied by the pier and formwork shall be over-excavated a minimum of 2' below the bottom of pier elevation. This area shall be partially filled with clean stone to a point approximately 6" below concrete grade. The preassembled form shall then be lowered into position. The form shall then be properly positioned, plumbed and secured to prevent movement during the pouring operation. Clean stone shall then be placed inside the form to ensure that concrete cannot leak out from under the form, as well as to attain the bottom of pier elevation.

Reinforcing steel may be incorporated in the form prior to setting, or it may be placed after the form has been set. A sufficient number of bar chairs shall be used to ensure that proper clearances are achieved.

Concrete shall then be placed through the use of an appropriate number of tremies for the width and height of the pier. The movement of tremies and recharging shall be limited as much as possible. Placement operations shall be in accordance with Subsection 502.3.5.3 of the Standard Specifications. When the concrete reaches an elevation above the water elevation, the surface shall be checked for any unsatisfactory materials which may have been forced upward as the concrete was being deposited. Any deleterious materials found shall be removed at that time from inside the form. Further concrete placement may then proceed using standard placement methods. Silt curtains or screens, as specified on the plans, shall be employed to limit downstream siltation. Displaced water shall be treated by filtration, settling basin or other approved means sufficient to reduce the cement content before being discharged into the stream.

After the formwork has been removed, the excavated area shall be backfilled to the original streambed elevation.

13.15 Design Examples

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E13-1 Hammerhead Pier Design Example

This example shows design calculations conforming to the ***AASHTO LRFD Bridge Design Specifications (Sixth Edition - 2013 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.

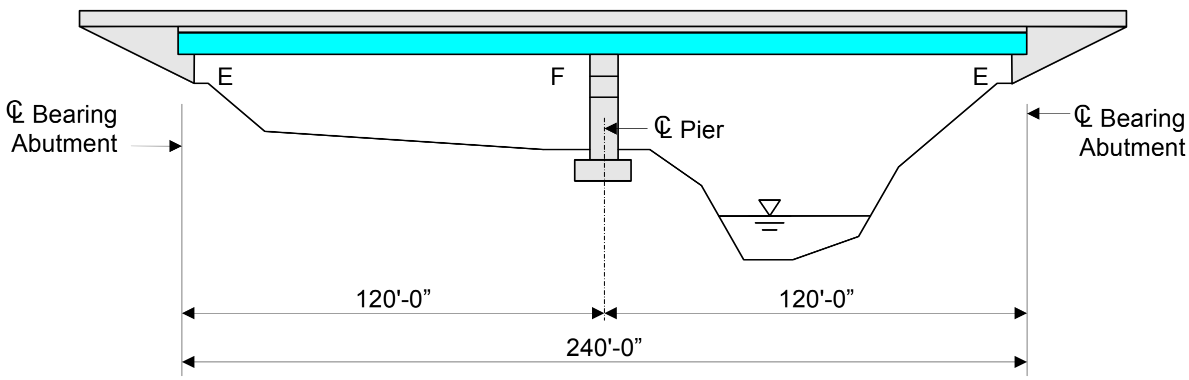

Figure E13-1.1-1
Bridge Elevation

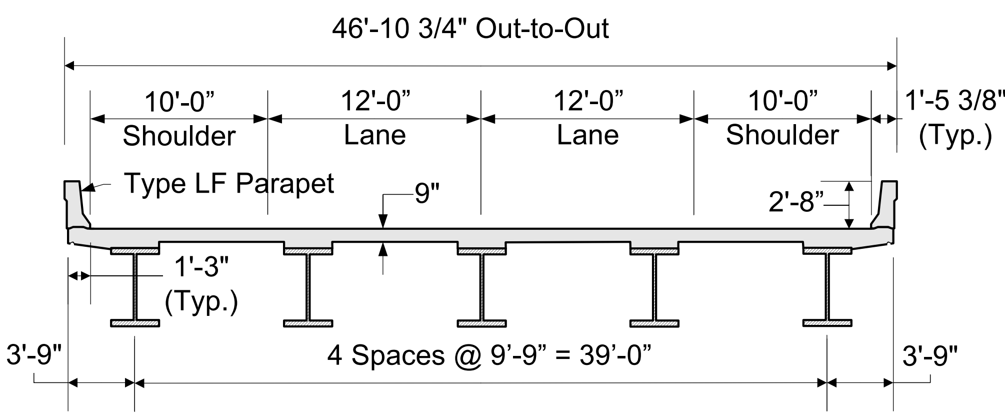

Figure E13-1.1-2
Bridge Cross Section

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

$b := 12$ inches

$a := \frac{As_{col} \cdot f_y}{0.85 \cdot f'_c \cdot b}$ $\boxed{a = 4.25}$ inches

$\beta_1 := 0.85$

$c := \frac{a}{\beta_1}$ $\boxed{c = 5.00}$ inches

$d_t := W_{col} \cdot 12 - Cover_{co} - 0.5 - \frac{bar\_dia10}{2}$ $\boxed{d_t = 44.37}$ 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**]

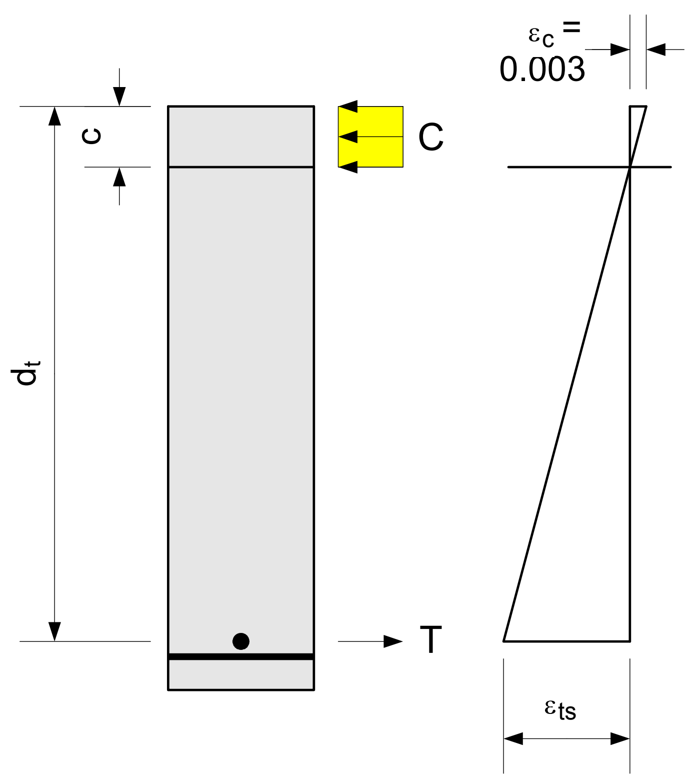

Figure E13-1.9-2
Strain Limit Tension Control Check

$\varepsilon_{ts} := \frac{\varepsilon_c}{c} \cdot (d_t - c)$ $\boxed{\varepsilon_{ts} = 0.016}$ $> \varepsilon_t = 0.005$

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 (f_{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 f_{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.

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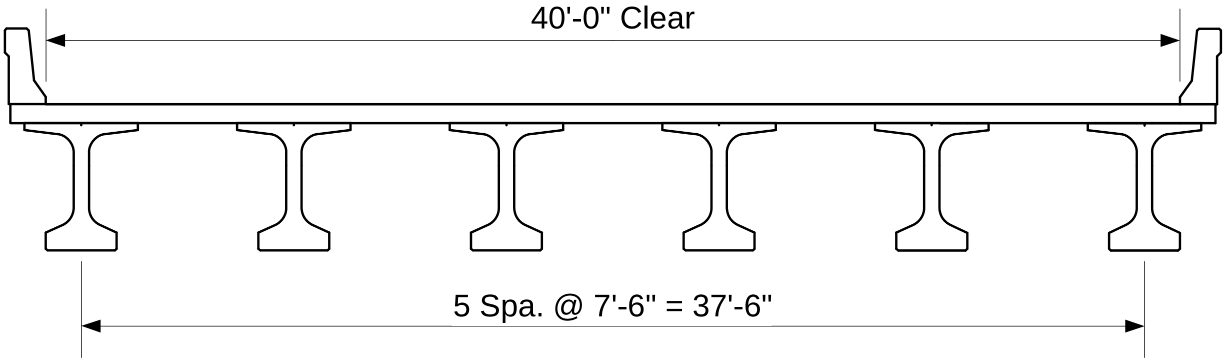
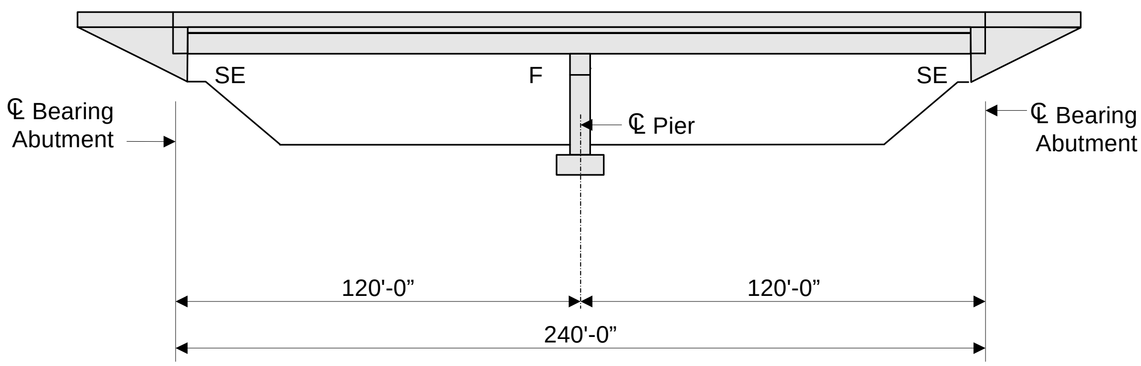
E13-2 Multi-Column Pier Design Example - LRFD

2 Span Bridge, 54W, LRFD Design



This pier is designed for the superstructure as detailed in example **E19-2**. This is a two-span prestressed girder grade separation structure. Semi-expansion bearings are located at the abutments, and fixed bearings are used at the pier.



E13-2.1 Obtain Design Criteria

This multi-column pier design example is based on ***AASHTO LRFD Bridge Design Specifications, (Sixth Edition - 2013 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

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho := \frac{As_{prov\_pos}}{b_w \cdot d_e} \qquad \boxed{\rho = 0.00778}$$

$$n := \mathrm{floor}\left(\frac{E_s}{E_c}\right) \qquad \boxed{n = 8}$$

$$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n \qquad \boxed{k = 0.3}$$

$$j := 1 - \frac{k}{3} \qquad \boxed{j = 0.9}$$

$$d_c := cover + Bar_D(bar_{stirrup}) + \frac{Bar_D(Bar_{No\_pos})}{2} \qquad \boxed{d_c = 3.69} \text{ in}$$

$$\boxed{Ms_{pos} = 1634} \text{ kip-ft}$$

$$f_s := \frac{Ms_{pos}}{As_{prov\_pos} \cdot j \cdot d_e} \cdot 12 \quad \le 0.6\, f_y \qquad \boxed{f_s = 36.24} \text{ ksi} \quad \text{approx.} = 0.6\, f_y \text{ O.K.}$$

The height of the section, h, is:

$$h := cap_H \cdot 12 \qquad \boxed{h = 48} \text{ in}$$

$$\beta := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta = 1.12}$$

$\gamma_e := 1.0$ for Class 1 exposure condition

$$S_{max} := \frac{700\gamma_e}{\beta \cdot f_s} - 2 \cdot d_c \qquad \boxed{S_{max} = 9.89} \text{ in}$$

$$\boxed{spa_{pos} = 4.64} \text{ in}$$

Is the bar spacing less than 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.

$$spa' := spa_{pos} \qquad \boxed{spa' = 4.64} \text{ in}$$

$$As' := Bar_A(Bar_{No\_pos}) \cdot n_{bars\_pos1} \qquad \boxed{As' = 9} \text{ in}^2$$

$$a' := \frac{As' \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$$ $\boxed{a' = 4.32}$ in

$$d_{e'} := cap_H \cdot 12 - cover - Bar_D(bar_{stirrup}) - \frac{Bar_D(Bar_{No\_pos})}{2}$$

$\boxed{d_e = 42.87}$ 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} less than $M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 1023}$ 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*Mu_{cut1}? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho' := \frac{As'}{b_w \cdot d_e}$$ $\boxed{\rho' = 0.00500}$

$$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n$$ $\boxed{k' = 0.25}$

$$j' := 1 - \frac{k'}{3}$$ $\boxed{j' = 0.92}$

$\boxed{Ms_{cut1} = 1051}$ kip-ft

$f_{s'} := \frac{Ms_{cut1}}{As' \cdot j' \cdot d_e} \cdot 12 \quad \leq 0.6\, f_y$ $\boxed{f_{s'} = 35.59}$ ksi $\leq 0.6\, f_y$ O.K.

$\boxed{\beta = 1.12}$

$\boxed{\gamma_e = 1}$

$S_{max'} := \frac{700 \gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c$ $\boxed{S_{max'} = 10.20}$ in

$\boxed{spa' = 4.64}$ in

Is the bar spacing less than $S_{max'}$? $\boxed{check = \text{"OK"}}$

The bars shall be extended past this cut off point for a distance not less than the following, **LRFD [5.11.1.2.1]**:

$\boxed{d_e = 42.87}$ in

$\boxed{15 \cdot Bar_D\left(Bar_{No\_pos}\right) = 16.92}$ in

$\boxed{\frac{col_{spa} \cdot 12}{20} = 10.95}$ in

$\boxed{BarExtend_{pos} = 42.87}$ in

The bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number $\boxed{\quad \rightarrow 9}$ bar with spacing less than 6-inches, is:

$l_{d\_9} := 5.083$ ft

$\boxed{cut_{pos} + \frac{BarExtend_{pos}}{12} = 14.27}$

$\boxed{0.4 \cdot col_{spa} + l_{d\_9} = 12.38}$

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.

$$spa_{neg} := \frac{b_w - 2 \cdot (cover - Bar_D(bar_{stirrup})) - Bar_D(Bar_{No\_neg})}{n_{bars\_neg} - 1}$$

$\boxed{spa_{neg} = 4.66}$ in

$$clear_{spa\_neg} := spa_{neg} - Bar_D(Bar_{No\_neg})$$

$\boxed{clear_{spa\_neg} = 3.66}$ in

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

$As_{prov\_neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg}$ $\boxed{As_{prov\_neg} = 7.07}$ in^2

$a_{neg} := \dfrac{As_{prov\_neg} \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$ $\boxed{a_{neg} = 3.39}$ in

$Mn_{neg} := As_{prov\_neg} \cdot f_y \cdot \left(d_e - \dfrac{a_{neg}}{2}\right) \cdot \dfrac{1}{12}$ $\boxed{Mn_{neg} = 1455}$ kip-ft

$Mr_{neg} := \phi_f \cdot Mn_{neg}$ $\boxed{Mr_{neg} = 1310}$ kip-ft

$\boxed{Mu_{neg} = 1174}$ 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]**:

$\boxed{M_{cr} = 1023}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg} = 1561}$ 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]**:

$\rho_{neg} := \dfrac{As_{prov\_neg}}{b_w \cdot d_{e\_neg}}$ $\boxed{\rho_{neg} = 0.00379}$

$\boxed{n = 8}$

$k_{neg} := \sqrt{(\rho_{neg} \cdot n)^2 + 2 \cdot \rho_{neg} \cdot n} - \rho_{neg} \cdot n$ $\boxed{k_{neg} = 0.22}$

$j_{neg} := 1 - \dfrac{k_{neg}}{3}$ $\boxed{j_{neg} = 0.93}$

$d_{c\_neg} := cover + Bar_D(bar_{stirrup}) + \dfrac{Bar_D(Bar_{No\_neg})}{2}$ $\boxed{d_{c\_neg} = 3.63}$ in

$\boxed{Ms_{neg} = 844}$ kip-ft

$f_{s\_neg} := \dfrac{Ms_{neg}}{As_{prov\_neg} \cdot j_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \le 0.6\, f_y$ $f_{s\_neg} = 34.8$ 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

$\boxed{spa_{neg} = 4.66}$ in

Is the bar spacing less than S_{max}? $\boxed{check = \text{"OK"}}$

E13-2.6.4 Negative Moment Reinforcement Cut Off Location

Cut 4 bars where the remaining 5 bars satisfy the moment diagram.

$n_{bars\_neg'} := 5$

$spa'_{neg} := spa_{neg} \cdot 2$ $\boxed{spa'_{neg} = 9.31}$ in

$As'_{neg} := Bar_A(Bar_{No\_neg}) \cdot n_{bars\_neg'}$ $\boxed{As'_{neg} = 3.93}$ in^2

$$a'_{neg} := \frac{As'_{neg} \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$$ $\boxed{a'_{neg} = 1.89}$ 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_{cut1} less than $M_{r'}$? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$\boxed{M_{cr} = 1023}$ kip-ft

$\boxed{1.33 \cdot Mu_{neg\_cut} = 767}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and $1.33*Mu_{cut1}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$$\rho'_{neg} := \frac{As'_{neg}}{b_w \cdot d_e} \qquad \boxed{\rho'_{neg} = 0.00218}$$

$$k'_{neg} := \sqrt{\left(\rho'_{neg} \cdot n\right)^2 + 2 \cdot \rho'_{neg} \cdot n} - \rho'_{neg} \cdot n \qquad \boxed{k'_{neg} = 0.17}$$

$$j'_{neg} := 1 - \frac{k'_{neg}}{3} \qquad \boxed{j'_{neg} = 0.94}$$

$\boxed{Ms_{neg\_cut} = 381}$ kip-ft

$$f_{s'\_neg} := \frac{Ms_{neg\_cut}}{As'_{neg} \cdot j'_{neg} \cdot d_{e\_neg}} \cdot 12 \quad \leq 0.6\ f_y \qquad \boxed{f_{s'\_neg} = 27.82} \text{ ksi} \quad \leq 0.6\ f_y \text{ O.K.}$$

$\boxed{\beta_{neg} = 1.12}$

$\boxed{\gamma_e = 1}$

$$S_{max'\_neg} := \frac{700\gamma_e}{\beta_{neg} \cdot f_{s'\_neg}} - 2 \cdot d_{c\_neg} \qquad \boxed{S_{max'\_neg} = 15.28} \text{ in}$$

$\boxed{spa'_{neg} = 9.31}$ in

Is the bar spacing less than $S_{max'}$? $\boxed{check = \text{"OK"}}$

The bars shall be extended past this cut off point for a distance not less than the following, **LRFD [5.11.1.2.3]**:

$\boxed{d_e = 42.87}$ in

$\boxed{12 \cdot Bar_D\left(Bar_{No\_neg}\right) = 12}$ in

$\boxed{\frac{\left(col_{spa} - col_w\right) \cdot 12}{16} = 10.69}$ in

$\boxed{BarExtend_{neg} = 42.87}$ in

These bars also must be extended past the point required for flexure the development length of the bar. From Chapter 9, **Table 9.9-1**, the development length for an epoxy coated number $\boxed{\rightarrow 8}$ "top" bar with spacing greater than 6-inches, is:

$l_{d\_8} := 3.25$ ft

The cut off location is determined by the following:

$\boxed{cut_{neg} - \frac{BarExtend_{neg}}{12} = 11.73}$ ft

$\boxed{col_{spa} - \frac{col_w}{2} - l_{d\_8} = 13}$ ft

Therefore, the cut off location is located at the following distance from the CL of the left column:

$\boxed{cutoff_{location} = 11.73}$ ft

By inspection, the remaining top mat reinforcement is adequate over the exterior columns. The inside face of the exterior column is located at:

$col_{face} := \frac{col_w}{2} \cdot \frac{1}{col_{spa}}$ $\boxed{col_{face} = 0.11}$ % along cap

$\boxed{Mu_{negative}\left(col_{face}\right) = -378.37}$ kip-ft

$\boxed{Ms_{negative}\left(col_{face}\right) = -229.74}$ kip-ft

E13-2.7 Reinforcement Summary

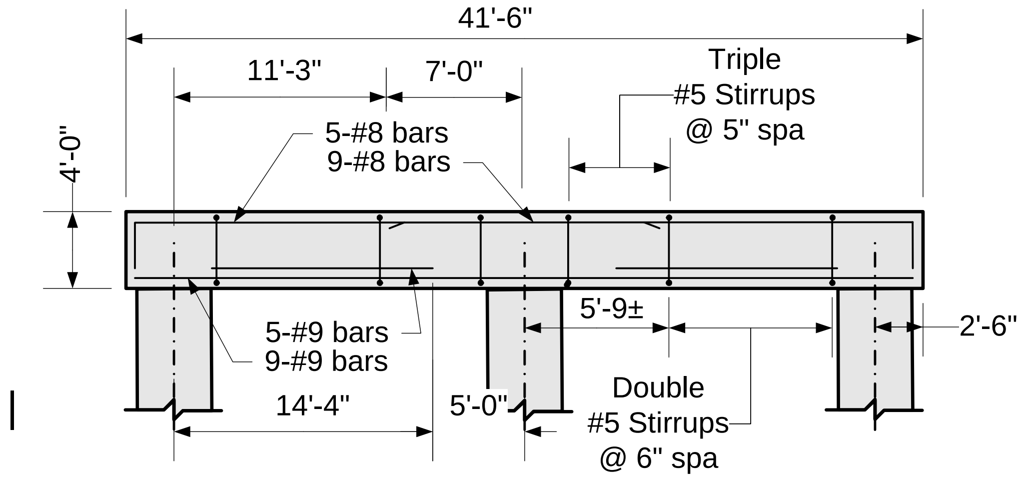


Figure E13-2.7-1
Cap Reinforcement - Elevation View

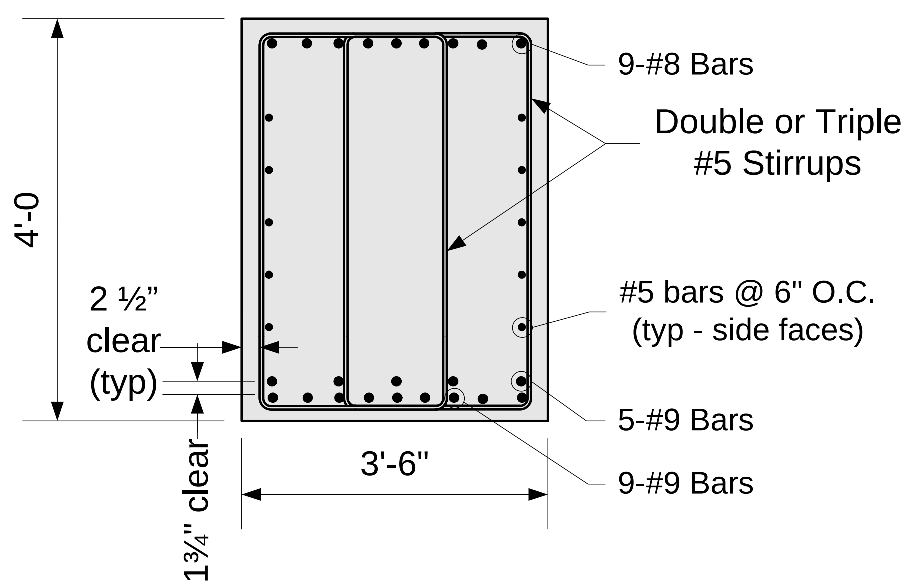


Figure E13-2.7-2
Cap Reinforcement - Section View

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Wall Type	Temp	Perm	Cost Effective Height (ft)	Req'd. ROW	Advantages	Disadvantages
Concrete Gravity		√	3-10	.5H-.7H	• Durable • Meets aesthetic requirement • Requires small quantity of select backfill	• High cost • May need deep foundation • Longer const. time
Reinforced CIP Cantilever		√	6-28	.4H-.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
Concrete Modular Block		√	3-8	.4H-.7H	• Does not require skilled labor or specialized equipment	• Height limitations
Metal Bin		√	6 -20	.4H-.7H	• Does not require skilled labor or special equipment	• Difficult to make height adjustment in the field
Concrete Crib		√	6-20	.4H-.7H	• Does not require skilled labor or specialized equipment	• Difficult to make height adjustment in the field
Gabion		√	6-20	.4H-.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	.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	.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	.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
Post & Panel	√	√	6-28	.2H-.5H	poor	• Easy construction • Readily available	• High cost • Deep foundation may be needed • Longer construction time
Tangent Pile		√	20 -60	.4H-.7H	good	• Adaptable to irregular layout • Can control wall stiffness	• High cost • Deep foundation may be needed • Longer construction
Secant Pile Wall		√	14-60	.4H-.7H	fair	• Adaptable to irregular layout • Can control wall stiffness	• Difficult to make height adjustment in the field • High cost
Anchored Wall	√	√	15-35	.4H-.7H	fair	• Rapid construction	• Difficult to make height adjustment in the field
Soil Nail Wall	√	√	6-20	.4H-.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. 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.

3. Wall Selection: Make wall type selection based on the steps 1 and 2 above and using the wall selection criteria discussed in 14.3.

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 proprietary walls, internal stability is the responsibility of the contractor/supplier

9. Repeat design steps 4 thru 8 if the required checks are not met.

14.4.2 Design Standards

Retaining wall systems shall be designed in conformance with the current *AASHTO Load and Resistance Factor Design Specifications* (AASHTO LRFD) and in accordance with the WisDOT Bridge Manual. Walls shall be designed to address all limit states.

Wall systems including rock walls and soil nail systems which are not specifically covered by the *AASHTO LRFD* specifications shall be designed using the hierarchy of guidelines presented in this chapter, Allowable Stress Design (ASD) or *AASHTO Load Factor Design* (LFD) methods or the design procedures developed based on standard engineering and/or industry practices. The guidelines presented in this chapter will prevail where interpretation differs. WisDOT's decision shall be final in those cases. The new specifications for the wall designs were implemented October 1st, 2010.

14.4.3 Design Life

All permanent retaining walls and components shall be designed for a minimum service life of 75 years. All temporary walls shall be designed for a period of 36 months or for the project specific duration, whichever is greater. The design of temporary wall systems is the responsibility of the contractor. The temporary walls shall meet all the safety requirements as that of a permanent wall except for corrosion and aesthetics.

14.4.4 Subsurface Exploration

Geotechnical exploration may be needed to explore the soil/rock properties for foundation, retained fill, and backfill soils for all retaining walls regardless of wall height. It is the designer's responsibility to ensure that pertinent soils information, loading conditions, foundation considerations, consolidation potential, settlement and external stability is provided for the wall design.

Before planning a subsurface investigation, it is recommended that any other available subsurface information such as geological or other maps or data available from previous subsurface investigations be studied. Subsurface investigation and analyses should be performed where necessary, in accordance with Chapter 10 - Geotechnical Investigation.

The investigations and analyses may be required to determine or establish the following:

- Nominal bearing pressure, consolidation properties, unit weight and shear strength (drained or undrained strength for fine grained soils) for foundation soils/rocks.
- Shear strength, and unit weight of selected backfill.
- Shear strength and unit weight of random fill or in-situ soil behind selected backfill or wall
- Location of water table

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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 Sixth Edition - 2013 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.

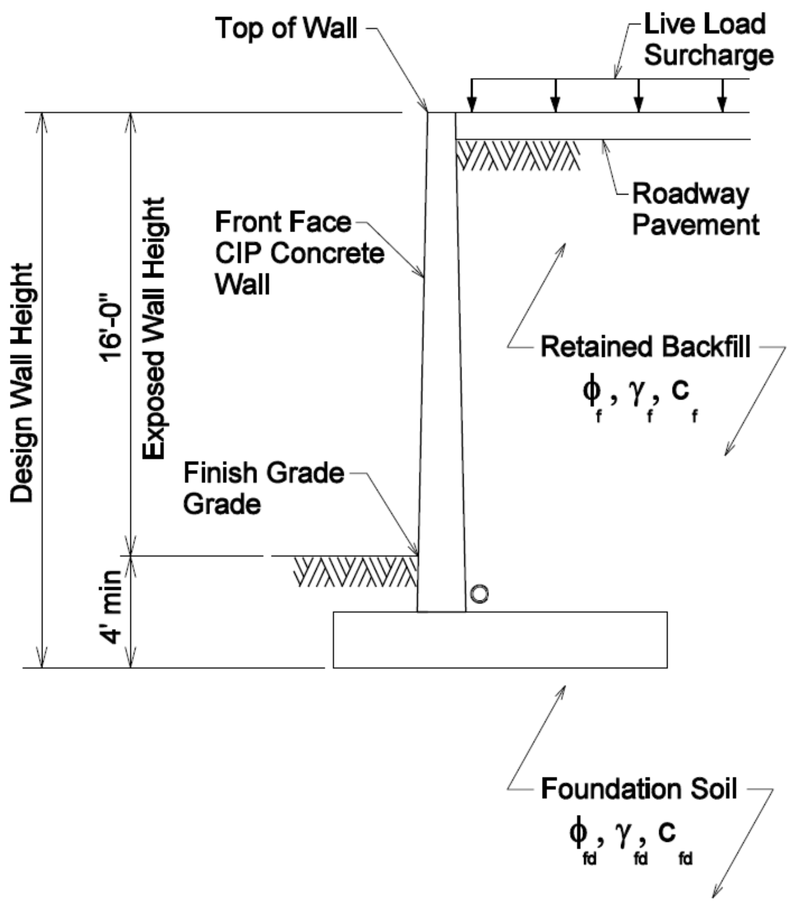

Figure E14-1.1-1
CIP Concrete Wall Adjacent to Highway

E14-1.5 Compute Bearing Resistance, q_R

Nominal bearing resistance, q_n **LRFD [Eq 10.6.3.1.2a-1]**

$$q_n = c_{fd} N_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B' N_{\gamma m} C_{w\gamma}$$

Compute the resultant location (distance from Point 'O' Figure E14-4.4-3)

$\Sigma M_R = MV\_Ib$ $\boxed{\Sigma M_R = 205.8}$ Summation of resisting moments for Strength Ib

$\Sigma M_O = MH\_Ib$ $\boxed{\Sigma M_O = 81.3}$ Summation of overturning moments for Strength Ib

$\Sigma V = V\_Ib$ $\boxed{\Sigma V = 29.3}$ Summation of vertical loads for Strength Ib

$x = \dfrac{\Sigma M_R - \Sigma M_O}{\Sigma V}$ Distance from Point "O" the resultant intersects the base $\boxed{x = 4.25}$ ft

Compute the wall eccentricity

$e = \dfrac{B}{2} - x$ $\boxed{e = 0.75}$ ft

Define the foundation layout

$B' = B - 2\,e$ Footing width $\boxed{B' = 8.5}$ ft

$L' = 90.0$ Footing length (Assumed) $\boxed{L' = 90.0}$ ft

$H' = H\_Ib$ Summation of horizontal loads for Strength Ib $\boxed{H' = 11.7}$ kip/ft

$V' = V\_Ib$ Summation of vertical loads for Strength Ib $\boxed{V' = 29.3}$ kip/ft

$D_f = 4.00$ Footing embedment

$\theta' = 90deg$ Direction of H' and V' resultant measured from wall backface **LRFD [Figure C10.6.3.1.2a-1]** $\boxed{\theta' = 90.0deg}$

Compute bearing capacity factors per **LRFD [Table 10.6.3.1.2a-1]**

$\phi_{fd} = 34.0deg$ $N_q = 29.4$ $N_c = 42.2$ $N_\gamma = 41.1$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**

Since the friction angle, ϕ_f, is > 0 the following equations are used:

$s_c = 1 + \left(\dfrac{B'}{L'}\right)\left(\dfrac{N_q}{N_c}\right)$ $\boxed{s_c = 1.07}$

$s_q = 1 + \left(\dfrac{B'}{L'}\tan(\phi_{fd})\right)$ $\boxed{s_q = 1.06}$

$s_\gamma = 1 - 0.4\left(\dfrac{B'}{L'}\right)$ $\boxed{s_\gamma = 0.96}$

Compute load inclination factors using **LRFD Equations [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]**

$$n = \frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}} \cos(\theta')^2 + \frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}} \sin(\theta')^2 \qquad \boxed{n = 1.91}$$

$$i_q = \left(1 - \frac{H'}{V' + c_{fd}\, B'\, L' \frac{1}{\tan(\phi_{fd})}}\right)^n \qquad \boxed{i_q = 0.38}$$

$$i_\gamma = \left(1 - \frac{H'}{V' + c_{fd}\, B'\, L' \frac{1}{\tan(\phi_{fd})}}\right)^{n+1} \qquad \boxed{i_\gamma = 0.23}$$

$$i_c = i_q - \left(\frac{1 - i_q}{N_q - 1}\right) \qquad \text{For } \phi_{fd} > 0: \qquad \boxed{i_c = 0.36}$$

Note: The use of load inclination factors shall be determined by the engineer.

Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. While it can be assumed that the soils above the footing are as competent as beneath the footing, the depth correction factor is taken as 1.0 since D_f/B is less than 1.0.

$$d_q = 1.00$$

Determine coefficients C_{wq} and $C_{w\gamma}$ assuming that the water depth is greater than 1.5 times the footing base plus the embedment depth per **LRFD [Table 10.6.3.1.2a-2]**

$$C_{wq} = 1.0 \qquad \text{where } D_w > 1.5B + D_f$$

$$C_{w\gamma} = 1.0 \qquad \text{where } D_w > 1.5B + D_f$$

Compute modified bearing capacity factors
LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]

$$N_{cm} = N_c\, s_c\, i_c \qquad \boxed{N_{cm} = 16.0}$$

$$N_{qm} = N_q\, s_q\, d_q\, i_q \qquad \boxed{N_{qm} = 11.8}$$

$$N_{\gamma m} = N_\gamma\, s_\gamma\, i_\gamma \qquad \boxed{N_{\gamma m} = 9.0}$$

Compute nominal bearing resistance, q_n, **LRFD [Eq 10.6.3.1.2a-1]**

$$q_n = c_{fd}\, N_{cm} + \gamma_{fd}\, D_f\, N_{qm}\, C_{wq} + 0.5\, \gamma_{fd}\, B'\, N_{\gamma m}\, C_{w\gamma} \qquad \boxed{q_n = 10.25} \text{ ksf/ft}$$

Compute factored bearing resistance, q_R, **LRFD [Eq 10.6.3.1.1]**

$$\phi_b = 0.55$$

$$q_R = \phi_b\, q_n \qquad \boxed{q_R = 5.64} \text{ ksf/ft}$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 21.0} \text{ in}$$

$$d_{v2} = 0.9\ d_s \qquad \boxed{d_{v2} = 19.5} \text{ in}$$

$$d_{v3} = 0.72\ D\ 12 \qquad \boxed{d_{v3} = 17.3} \text{ in}$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 21.0} \text{ in}$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$$\beta = 2.0$$

$$V_c = 0.0316\ \beta\ \sqrt{f'_c}\ b\ d_v$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 29.8} \text{ kip/ft}$$

$$V_{n2} = 0.25\ f'_c\ b\ d_v \qquad \boxed{V_{n2} = 220.4} \text{ kip/ft}$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 29.8} \text{ kip/ft}$$

$$V_r = \phi_v\ V_n \qquad \boxed{V_r = 26.8} \text{ kip/ft}$$

$$\boxed{V_u = 21.9} \text{ kip/ft}$$

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-1.7.1.2 Evaluate Heel Flexural Strength

$$V_u = 21.9 \text{ kip/ft}$$

$$M_u = V_u\ \frac{C}{2} \qquad \boxed{M_u = 47.9} \text{ kip-ft/ft}$$

Calculated the capacity of the heel in flexure at the face of the stem:

$$M_n = A_s\ f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12} \qquad \boxed{M_n = 79.2} \text{ kip-ft/ft}$$

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a}{\beta_1} \qquad \boxed{c = 1.49} \text{ in}$$

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_s}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_s}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Note:	if $\phi_F = 0.75$	Section is compression-controlled
	if $0.75 < \phi_F < 0.90$	Section is in transition
	if $\phi_F = 0.90$	Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F M_n$$

$\boxed{M_r = 71.2}$ kip-ft/ft

$\boxed{M_u = 47.9}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12}\, b\,(D\ 12)^3$$ $\boxed{I_g = 13824}$ in^4

$$y_t = \frac{1}{2}\, D\ 12$$ $\boxed{y_t = 12.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1152}$ in^3

$M_{cr} = \gamma_3(\gamma_1 f_r)\, S_c$ therefore, $M_{cr} = 1.1\, f_r\, S_c$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement <u>for A615, Grade 60 reinforcement</u>

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$$ $\boxed{M_{cr} = 73.1}$ kip-ft/ft

$\boxed{1.33\ M_u = 63.7}$ 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)

$Bar_A = 0.31$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 0.41} \; in^2/ft$$

$$d_s = D\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 20.7} \; in$$

$$a = \frac{A_s\,f_y}{0.85\,f'_c\,b} \qquad \boxed{a = 0.7} \; in$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 20.3} \; in$$

$$d_{v2} = 0.9\,d_s \qquad \boxed{d_{v2} = 18.6} \; in$$

$$d_{v3} = 0.72\,D\,12 \qquad \boxed{d_{v3} = 17.3} \; in$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 20.3} \; in$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$$\beta = 2.0$$

$$V_c = 0.0316\,\beta\,\sqrt{f'_c}\,b\,d_v$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 28.9} \; kip/ft$$

$$V_{n2} = 0.25\,f'_c\,b\,d_v \qquad \boxed{V_{n2} = 213.6} \; kip/ft$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 28.9} \; kip/ft$$

$$V_r = \phi_v\,V_n \qquad \boxed{V_r = 26.0} \; kip/ft$$

$$\boxed{V_u = 13.2} \; kip/ft$$

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

E14-1.7.2.2 Evaluate Toe Flexural Strength

$V_u = 13.2$ kip/ft

$M_u = V_u \frac{A}{2}$ $\boxed{M_u = 23.2}$ kip-ft/ft

Calculated the capacity of the toe in flexure at the face of the stem:

$M_n = A_s f_y \left(d_s - \frac{a}{2}\right) \frac{1}{12}$ $\boxed{M_n = 42.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \frac{a}{\beta_1}$ $\boxed{c = 0.82}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 37.8}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.692}$ ksi

$I_g = \frac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 13824}$ in^4

$y_t = \frac{1}{2} D\ 12$ $\boxed{y_t = 12.00}$ in

$S_c = \frac{I_g}{y_t}$ $\boxed{S_c = 1152}$ in^3

$M_{cr} = 1.1\ f_r\ S_c \frac{1}{12}$ from E14-1.7.1.2 $\boxed{M_{cr} = 73.1}$ kip-ft/ft

$\boxed{1.33\ M_u = 30.8}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33{*}M_u$?

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

E14-1.7.3 Evaluate Stem Strength

Unfactored Stem Horizontal Loads and Moments:

$H_1 = \gamma_f\, h_{eq}\, (h' - t)\, k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_1 = 1.2}$ kip/ft

$H_2 = \frac{1}{2} \gamma_f\, (h' - t)^2\, k_a \cos(90\ deg - \theta + \delta)$ $\boxed{H_2 = 5.0}$ kip/ft

$M_1 = H_1 \left(\frac{h' - t}{2}\right)$ $\boxed{M_1 = 10.0}$ kip-ft/ft

$M_2 = H_2 \left(\frac{h' - t}{3}\right)$ $\boxed{M_2 = 28.4}$ kip-ft/ft

Factored Stem Horizontal Loads and Moments:

for **Strength Ib**:

$H_{u1} = 1.75\ H_1 + 1.50\ H_2$ $\boxed{H_{u1} = 9.6}$ kip/ft

$M_{u1} = 1.75\ M_1 + 1.50\ M_2$ $\boxed{M_{u1} = 60.0}$ kip-ft/ft

for **Service I**:

$H_{u3} = 1.00\ H_1 + 1.00\ H_2$ $\boxed{H_{u3} = 6.2}$ kip/ft

$M_{u3} = 1.00\ M_1 + 1.00\ M_2$ $\boxed{M_{u3} = 38.4}$ kip-ft/ft

E14-1.7.3.1 Evaluate Stem Shear Strength at Footing

$V_u = H_{u1}$ $\boxed{V_u = 9.6}$ kip/ft

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.8.3.3]**

$V_{n1} = V_c$ **LRFD [Eq 5.8.3.3-1]**

where: $V_c = 0.0316\ \beta \sqrt{f'_c}\ b_v\ d_v$

$V_{n2} = 0.25\ f'_c\ b_v\ d_v$ **LRFD [Eq 5.8.3.3-2]**

Compute the shear resistance due to concrete, V_c :

$cover = 2.0$ in

$s = 10.0$ in (bar spacing)

$Bar_{No} = 8$ (transverse bar size)

$Bar_D = 1.00$ in (transverse bar diameter)

$Bar_A = 0.79$ in^2 (transverse bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_s = 0.95} \text{ in}^2\text{/ft}$$

$$d_s = T_b\,12 - cover - \frac{Bar_D}{2} \qquad \boxed{d_s = 23.0} \text{ in}$$

$$a = \frac{A_s\,f_y}{0.85\,f'_c\,b} \qquad \boxed{a = 1.6} \text{ in}$$

$$d_{v1} = d_s - \frac{a}{2} \qquad \boxed{d_{v1} = 22.2} \text{ in}$$

$$d_{v2} = 0.9\,d_s \qquad \boxed{d_{v2} = 20.7} \text{ in}$$

$$d_{v3} = 0.72\,T_b\,12 \qquad \boxed{d_{v3} = 18.4} \text{ in}$$

$$d_v = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_v = 22.2} \text{ in}$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2}

$$\beta = 2.0$$

$$V_c = 0.0316\,\beta\,\sqrt{f'_c}\,b\,d_v$$

$$V_{n1} = V_c \qquad \boxed{V_{n1} = 31.5} \text{ kip/ft}$$

$$V_{n2} = 0.25\,f'_c\,b\,d_v \qquad \boxed{V_{n2} = 233.1} \text{ kip/ft}$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 31.5} \text{ kip/ft}$$

$$V_r = \phi_v\,V_n \qquad \boxed{V_r = 28.4} \text{ kip/ft}$$

$$\boxed{V_u = 9.6} \text{ kip/ft}$$

Is V_u less than V_r? $\boxed{check = \text{"OK"}}$

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

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.69}$ ksi

$$I_g = \frac{1}{12} b \left(T_b\ 12\right)^3$$ $\boxed{I_g = 16581}$ in^4

$$y_t = \frac{1}{2} T_b\ 12$$ $\boxed{y_t = 12.8}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1301}$ in^3

$M_{cr\_s} = 1.1\ f_r\ S_c\ \frac{1}{12}$ from E14-1.7.1.2 $\boxed{M_{cr\_s} = 82.5}$ kip-ft/ft

$\boxed{1.33\ M_u = 79.9}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = "OK"}$

Check the Service Ib crack control requirements in accordance with **LRFD [5.7.3.4]**

$\rho = \frac{A_s}{d_s\ b}$ $\boxed{\rho = 0.00343}$

$n = \frac{E_s}{E_c}$ $\boxed{n = 8.09}$

$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n$ $\boxed{k = 0.210}$

$j = 1 - \frac{k}{3}$ $\boxed{j = 0.930}$

$d_c = cover + \frac{Bar_D}{2}$ $\boxed{d_c = 2.5}$ in

$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12 \le 0.6\ f_y$ $\boxed{f_{ss} = 22.7}$ ksi $\le 0.6\ f_y$ O.K.

$h = T_b\ 12$

$\beta_s = 1 + \frac{d_c}{0.7\ (h - d_c)}$ $\boxed{\beta_s = 1.2}$

$\gamma_e = 1.0$ for Class 1 exposure

$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c$ $\boxed{s_{max} = 21.7}$ in

$\boxed{s = 10.0}$ in

Is the bar spacing less than s_{max}? $\boxed{check = "OK"}$

E14-1.7.3.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-1.7.4 Temperature and Shrinkage Steel

E14-1.7.4.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required. However, #4 bars at 18" o.c. (max) are placed longitudinally to serve as spacers.

E14-1.7.4.2 Temperature and Shrinkage Steel of Stem

The stem will be exposed to daily temperature changes. In accordance with **LRFD [5.10.8]** the stem shall provide temperature and shrinkage steel on each face and in each direction as calculated below:

$s = 18.0$ in (bar spacing)

$Bar_{No} = 4$ (bar size)

$Bar_D = 0.50$ in (temperature and shrinkage bar diameter)

$Bar_A = 0.20$ in^2 (temperature and shrinkage bar area)

$$A_s = \frac{Bar_A}{\frac{s}{12}}$$ (temperature and shrinkage provided)

$\boxed{A_s = 0.13}$ in^2/ft

$b_s = (H - D)\ 12$ least width of stem $\boxed{b_s = 216.0}$ in

$h_s = T_t\ 12$ least thickness of stem $\boxed{h_s = 12.0}$ in

$$A_{ts} = \frac{1.3\ b_s\ h_s}{2\left(b_s + h_s\right)\ f_y}$$ Area of reinforcement per foot, on each face and in each direction

$\boxed{A_{ts} = 0.12}$ in^2/ft

Is $0.11 \le A_s \le 0.60$? $\boxed{check = \text{"OK"}}$

Is $A_s > A_{ts}$? $\boxed{check = \text{"OK"}}$

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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 Sixth Edition - 2013 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.

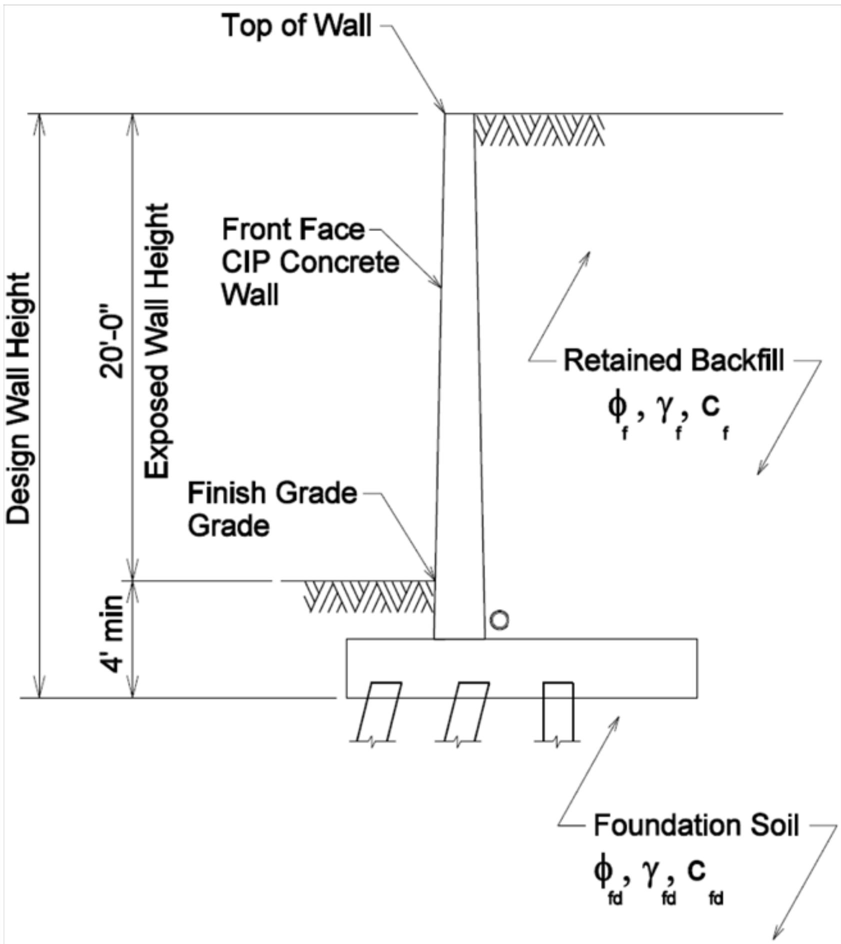


Figure E14-4.1-1
CIP Concrete Wall on Piles

Determine the location of the pile critical perimeter. Assume that the critical section is outside of the footing and only include the portion of the shear perimeter is located within the footing:

$$b_{o\_xx} = 1.25\ 12 + \frac{B_{xx}}{2} + \frac{d_{v\_toe}}{2} \qquad \boxed{b_{o\_xx} = 32.5} \text{ in}$$

$$b_{o\_yy} = 1.25\ 12 + \frac{B_{yy}}{2} + \frac{d_{v\_toe}}{2} \qquad \boxed{b_{o\_yy} = 32.3} \text{ in}$$

$$\beta_{c\_pile} = \frac{b_{o\_xx}}{b_{o\_yy}} \qquad \boxed{\beta_{c\_pile} = 1.004} \text{ in}$$

$$b_{o\_pile} = b_{o\_xx} + b_{o\_yy} \qquad \boxed{b_{o\_pile} = 64.8} \text{ in}$$

Nominal shear resistance, V_n, is taken as the lesser of V_{n1} and V_{n2} **LRFD [5.13.3.6.3]**

$$V_{n1} = \left(0.063 + \frac{0.126}{\beta_{c\_pile}}\right)\sqrt{f'_c}\ b_{o\_pile}\ d_{v\_toe} \qquad \boxed{V_{n1} = 523.1} \text{ kip/ft}$$

$$V_{n2} = 0.126\sqrt{f'_c}\ b_{o\_pile}\ d_{v\_toe} \qquad \boxed{V_{n2} = 349.7} \text{ kip/ft}$$

$$V_n = \min(V_{n1}, V_{n2}) \qquad \boxed{V_n = 349.7} \text{ kip/ft}$$

$$V_r = \phi_v V_n \qquad \boxed{V_r = 314.7} \text{ kip/ft}$$

$$\boxed{V_u = 150.2} \text{ kip/ft}$$

Is V_u less than V_r? $\boxed{\text{check} = \text{"OK"}}$

E14-4.7.1.3 Evaluate Top Transverse Reinforcement Strength

Top transverse reinforcement strength is determined by assuming the heel acts as a cantilever member supporting its own weight and loads acting above it. Pile reactions may be used to decrease this load.

For **Strength Ib**:

$$V_u = 1.25\left(\frac{C}{B}V_4\right) + 1.35\left(V_7 + V_8 + V_9\right) + 1.75\left(V_{10}\right) + 1.50\left(V_{11}\right)$$

$$\boxed{V_u = 27.0} \text{ kip/ft}$$

$$M_u = V_u \frac{C}{2} \qquad \boxed{M_u = 66.3} \text{ kip-ft/ft}$$

Calculated the capacity of the heel in flexure at the face of the stem:

$$M_n = A_{s\_heel}\ f_y\left(d_{s\_heel} - \frac{a\_heel}{2}\right)\frac{1}{12} \qquad \boxed{M_n = 107.6} \text{ kip-ft/ft}$$

Calculate the flexural resistance factor ϕ_F:

$\beta_1 = 0.85$

$c = \dfrac{a\_heel}{\beta_1}$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \dfrac{d_{s\_heel}}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_{s\_heel}}{c} - 1\right) & \text{if } \dfrac{5}{3} \le \dfrac{d_{s\_heel}}{c} \le \dfrac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi, **LRFD [5.5.4.2.1], [Table C5.7.2.1-1]**

Note: if ϕ_F = 0.75 Section is compression-controlled
if 0.75 < ϕ_F < 0.90 Section is in transition
if ϕ_F = 0.90 Section is tension-controlled

Calculate the flexural factored resistance, M_r:

$M_r = \phi_F M_n$ $\boxed{M_r = 96.8}$ kip-ft/ft

$\boxed{M_u = 66.3}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$f_r = 0.37\sqrt{f'_c}$ $\boxed{f_r = 0.692}$ ksi

$I_g = \dfrac{1}{12} b (D\ 12)^3$ $\boxed{I_g = 27000}$ in^4

$y_t = \dfrac{1}{2} D\ 12$ $\boxed{y_t = 15.00}$ in

$S_c = \dfrac{I_g}{y_t}$ $\boxed{S_c = 1800}$ in^3

$M_{cr} = \gamma_3(\gamma_1 f_r) S_c$ therefore, $M_{cr} = 1.1\ f_r\ S_c$

Where:

$\gamma_1 = 1.6$ flexural cracking variability factor

$\gamma_3 = 0.67$ ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement for A615, Grade 60 reinforcement

$$M_{cr} = 1.1\, f_r\, S_c\, \frac{1}{12}$$ $\boxed{M_{cr} = 114.2}$ kip-ft/ft

$\boxed{1.33\, M_u = 88.2}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33 \cdot M_u$? $\boxed{check = \text{"OK"}}$

E14-4.7.1.4 Evaluate Bottom Transverse Reinforcement Strength

Bottom transverse reinforcement strength is determined by using the maximum pile reaction.

Determine the moment arms

$$arm\_v1 = A - y_{p1}$$ $\boxed{arm\_v1 = 3.5}$ ft

$$arm\_v2 = A - y_{p2}$$ $\boxed{arm\_v2 = 0.8}$ ft

Determine the moment for **Strength Ia**:

$$V_{u\_1a} = P_{U1a}\, NP_1$$ $\boxed{V_{u\_1a} = 9.8}$ kip/ft

$$V_{u\_2a} = P_{U2a}\, NP_2$$ $\boxed{V_{u\_2a} = 9.5}$ kip/ft

$$M_{u\_Ia} = V_{u\_1a}\, arm\_v1 + V_{u\_2a}\, arm\_v2$$ $\boxed{M_{u\_Ia} = 41.6}$ kip-ft/ft

Determine the moment for **Strength Ib**:

$$V_{u\_1b} = P_{U1b}\, NP_1$$ $\boxed{V_{u\_1b} = 6.8}$ kip/ft

$$V_{u\_2b} = P_{U2b}\, NP_2$$ $\boxed{V_{u\_2b} = 12.5}$ kip/ft

$$M_{u\_Ib} = V_{u\_1b}\, arm\_v1 + V_{u\_2b}\, arm\_v2$$ $\boxed{M_{u\_Ib} = 33.3}$ kip-ft/ft

Determine the design moment:

$$M_u = \max\left(M_{u\_Ia}, M_{u\_Ib}\right)$$ $\boxed{M_u = 41.6}$ kip-ft/ft

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_{s\_toe}\, f_y \left(d_{s\_toe} - \frac{a\_toe}{2}\right) \frac{1}{12}$$ $\boxed{M_n = 91.6}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{vmatrix} 0.75 \quad \text{if} \quad \dfrac{d_{s\_toe}}{c} < \dfrac{5}{3} \\ 0.65 + 0.15\left(\dfrac{d_{s\_toe}}{c} - 1\right) \quad \text{if} \quad \dfrac{5}{3} \le \dfrac{d_{s\_toe}}{c} \le \dfrac{8}{3} \\ 0.90 \quad \text{otherwise} \end{vmatrix}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi,
LRFD [5.5.4.2.1], [Table C5.7.2.1-1]

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F M_n$$ $\boxed{M_r = 82.4}$ kip-ft/ft

$\boxed{M_u = 41.6}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12} b (D\ 12)^3$$ $\boxed{I_g = 27000}$ in^4

$$y_t = \frac{1}{2} D\ 12$$ $\boxed{y_t = 15.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1800}$ in^3

$$M_{cr} = 1.1\, f_r\, S_c \frac{1}{12}$$ from E14-4.7.1.3 $\boxed{M_{cr} = 114.2}$ kip-ft/ft

$\boxed{1.33\ M_u = 55.3}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = "OK"}$

E14-4.7.1.5 Evaluate Longitudinal Reinforcement Strength

The structural design of the longitudinal reinforcement, assuming the footing acts as a continuous beam over pile supports, is calculated using the maximum pile reactions.

Compute the effective shear depth, d_v ,for the longitudinal reinforcement

$cover = 6.0$ in

$s = 12.0$ in (bar spacing)

$Bar_{No} = 5$ (longitudinal bar size)

$Bar_D = 0.625$ in (longitudinal bar diameter)

$Bar_A = 0.310$ in² (longitudinal bar area)

$$A_{s\_long} = \frac{Bar_A}{\frac{s}{12}} \qquad \boxed{A_{s\_long} = 0.31} \text{ in}^2\text{/ft}$$

$$d_s = D\ 12 - cover - Bar_{D\_toe} - \frac{Bar_D}{2} \qquad \boxed{d_s = 22.8} \text{ in}$$

$$a\_long = \frac{A_{s\_long}\ f_y}{0.85\ f'_c\ b} \qquad \boxed{a\_long = 0.5} \text{ in}$$

$$d_{v1} = d_s - \frac{a\_long}{2} \qquad \boxed{d_{v1} = 22.6} \text{ in}$$

$$d_{v2} = 0.9\ d_s \qquad \boxed{d_{v2} = 20.5} \text{ in}$$

$$d_{v3} = 0.72\ D\ 12 \qquad \boxed{d_{v3} = 21.6} \text{ in}$$

$$d_{v\_long} = \max(d_{v1}, d_{v2}, d_{v3}) \qquad \boxed{d_{v\_long} = 22.6} \text{ in}$$

Calculate the design moment using a uniform vertical load:

$$L_{pile} = \max(P_1, P_2, P_3) \qquad \boxed{L_{pile} = 8.0} \text{ ft}$$

$$w_u = \frac{V\_Ib}{B} \qquad \boxed{w_u = 3.2} \text{ kip/ft/ft}$$

$$M_u = \frac{w_u\ L_{pile}^{\ 2}}{10} \qquad \boxed{M_u = 20.3} \text{ kip-ft/ft}$$

Calculated the capacity of the toe in flexure at the face of the stem:

$$M_n = A_{s\_long} f_y \left(d_s - \frac{a\_long}{2}\right) \frac{1}{12}$$ $\boxed{M_n = 35.0}$ kip-ft/ft

Calculate the flexural resistance factor ϕ_F:

$$\beta_1 = 0.85$$

$$c = \frac{a\_toe}{\beta_1}$$ $\boxed{c = 1.58}$ in

$$\phi_F = \begin{cases} 0.75 & \text{if } \frac{d_s}{c} < \frac{5}{3} \\ 0.65 + 0.15\left(\frac{d_s}{c} - 1\right) & \text{if } \frac{5}{3} \le \frac{d_s}{c} \le \frac{8}{3} \\ 0.90 & \text{otherwise} \end{cases}$$

$\boxed{\phi_F = 0.90}$

based on f_y = 60 ksi,
LRFD [5.5.4.2.1], [Table C5.7.2.1-1]

Calculate the flexural factored resistance, M_r:

$$M_r = \phi_F M_n$$ $\boxed{M_r = 31.5}$ kip-ft/ft

Is M_u less than M_r? $\boxed{check = "OK"}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.692}$ ksi

$$I_g = \frac{1}{12} b (D\ 12)^3$$ $\boxed{I_g = 27000}$ in^4

$$y_t = \frac{1}{2} D\ 12$$ $\boxed{y_t = 15.00}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1800}$ in^3

$$M_{cr} = 1.1\, f_r\, S_c \frac{1}{12}$$ from E14-4.7.1.3 $\boxed{M_{cr} = 114.2}$ kip-ft/ft

$\boxed{1.33\ M_u = 27.1}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and 1.33*M_u? $\boxed{check = "OK"}$

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

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b}$$ $\boxed{a = 1.7}$ 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:

$\beta_1 = 0.85$

$$c = \frac{a}{\beta_1}$$ $\boxed{c = 1.98}$ 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:

$$M_r = \phi_F M_n$$ $\boxed{M_r = 111.2}$ kip-ft/ft

$\boxed{M_u = 94.0}$ kip-ft/ft

Is M_u less than M_r? $\boxed{\text{check} = \text{"OK"}}$

Check the minimum reinforcement limits in accordance with **LRFD [5.7.3.3.2]**:

$$f_r = 0.37\sqrt{f'_c}$$ $\boxed{f_r = 0.69}$ ksi

$$I_g = \frac{1}{12}\, b\, (T_b\, 12)^3$$ $\boxed{I_g = 22247}$ in^4

$$y_t = \frac{1}{2}\, T_b\, 12$$ $\boxed{y_t = 14.1}$ in

$$S_c = \frac{I_g}{y_t}$$ $\boxed{S_c = 1582}$ in^3

$$M_{cr\_s} = 1.1\, f_r\, S_c\, \frac{1}{12}$$ from E14-4.7.1.3

$\boxed{M_{cr\_s} = 100.4}$ kip-ft/ft

$\boxed{1.33\, M_u = 125.0}$ kip-ft/ft

Is M_r greater than the lesser value of M_{cr} and $1.33*M_u$? $\boxed{\text{check} = \text{"OK"}}$

Check the Service Ib crack control requirements in accordance with **LRFD [5.7.3.4]**

$$\rho = \frac{A_s}{d_s\ b} \qquad \boxed{\rho = 0.00326}$$

$$n = \frac{E_s}{E_c} \qquad \boxed{n = 8.09}$$

$$k = \sqrt{(\rho\ n)^2 + 2\ \rho\ n} - \rho\ n \qquad \boxed{k = 0.205}$$

$$j = 1 - \frac{k}{3} \qquad \boxed{j = 0.932}$$

$$d_c = cover + \frac{Bar_D}{2} \qquad \boxed{d_c = 2.6}\ \text{in}$$

$$f_{ss} = \frac{M_{u3}}{A_s\ j\ d_s}\ 12 \le 0.6\ f_y \qquad \boxed{f_{ss} = 31.0}\ \text{ksi} \quad \le 0.6\ f_y \ \text{O.K.}$$

$$h = T_b\ 12$$

$$\beta_s = 1 + \frac{d_c}{0.7\left(h - d_c\right)} \qquad \boxed{\beta_s = 1.1}$$

$\gamma_e = 1.00$ for Class 1 exposure

$$s_{max} = \frac{700\ \gamma_e}{\beta_s\ f_{ss}} - 2\ d_c \qquad \boxed{s_{max} = 14.6}\ \text{in}$$

$$\boxed{s = 12.0}\ \text{in}$$

Is the bar spacing less than s_{max}? $\boxed{check = \text{"OK"}}$

E14-4.7.2.3 Transfer of Force at Base of Stem

Specification requires that the transfer of lateral forces from the stem to the footing be in accordance with the shear-transfer provisions of **LRFD [5.8.4]**. That calculation will not be presented. Refer to E13-1.9.3 for a similar computation.

E14-4.7.3 Temperature and Shrinkage Steel

E14-4.7.3.1 Temperature and Shrinkage Steel for Footing

The footing will not be exposed to daily temperature changes. Thus temperature and shrinkage steel is not required.

Table of Contents

17.1 Design Method

17.1.1 Design Requirements

All new structures and deck replacements are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*. Bridge rehabilitations and widenings are to be designed using either LFD or LRFD, at the designer's option.

LRFD utilizes load combinations called limit states which represent the various loading conditions which structural materials must be able to withstand. Limit states have been established in four major categories – strength, service, fatigue and extreme event. Different load combinations are used to analyze a structure for certain responses such as deflections, permanent deformations, ultimate strength and inelastic responses without failure. When all applicable limit states and combinations are satisfied, a structure is deemed acceptable under the LRFD design philosophy.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

17.1.2 Rating Requirements

All new structures and deck replacements are rated for *AASHTO LRFD* (HL-93) live loads. Rating factors, RF, for inventory and operating rating are shown on the plans. Ratings will be based on *The Manual for* Bridge Evaluation, hereafter referred to as *AASHTO MBE*. See Chapter 45 – Bridge Rating for rating requirements. Existing ratings for rehabilitation projects where the final ratings will not change should be taken from HSI and placed on the final plans. See Section 6.2.2.3.4 for more information.

17.1.2.1 Standard Permit Design Check

New structures are also to be checked for the Wisconsin Standard Permit Vehicle (Wis-SPV). The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface. This truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the bridge, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM.

The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

See Chapter 45 – Bridge Rating for details about the Wisconsin Standard Permit Vehicle and calculating the maximum load for this permit vehicle.

17.2 LRFD Requirements

17.2.1 General

For superstructure member design, the component dimensions and the size and spacing of reinforcement shall be selected to satisfy the following equation for all appropriate limit states, as presented in **LRFD [1.3.2.1]**:

$$Q = \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

Where:

η_i	=	Load modifier (a function of η_D, η_R, and η_i)
γ_i	=	Load factor
Q_i	=	Force effect: moment, shear, stress range or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance: resistance of a component to force effects
R_r	=	Factored resistance = ϕR_n

17.2.2 WisDOT Policy Items

WisDOT policy items:

Set the value of the load modifier, η_i (see **LRFD [1.3.2.1]**), and its factors, η_D, η_R and η_I, all equal to 1.00.

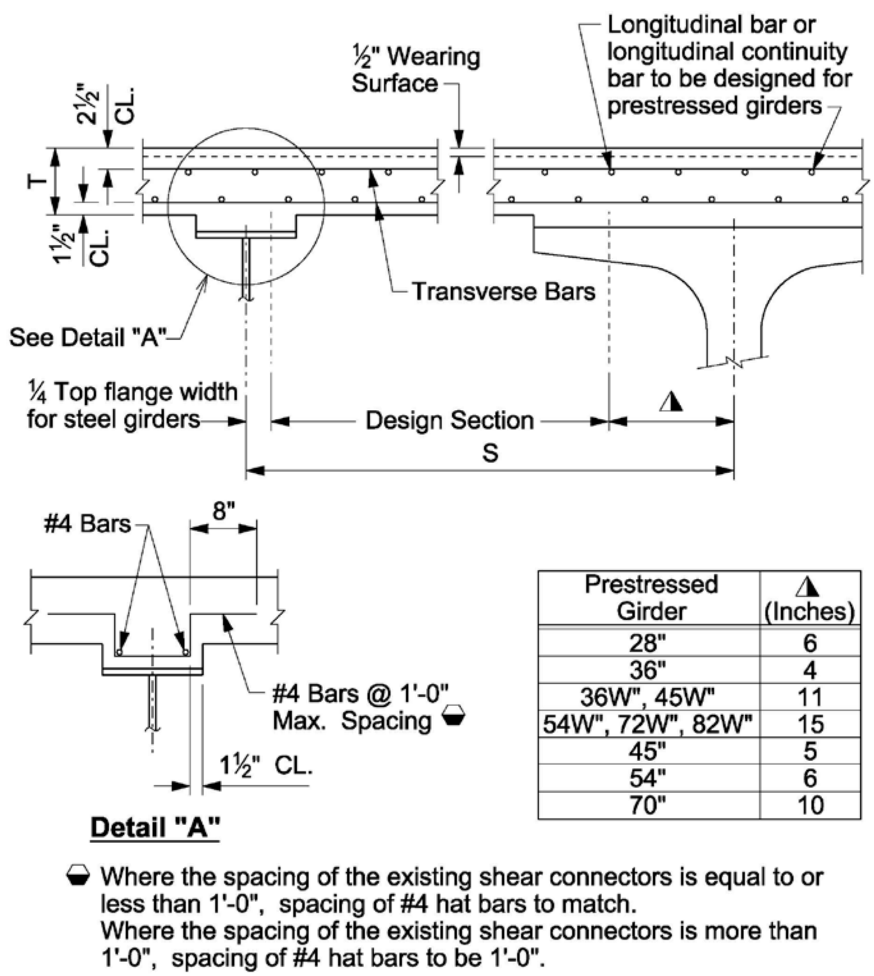
Ignore any influence of ADTT on multiple presence factor, m, in **LRFD [Table 3.6.1.1.2-1]** that would reduce force effects.

17.2.3 Limit States

The following limit states (as defined in **LRFD [3.4.1]**) are utilized by WisDOT in the design of bridge superstructures.

The distance from the centerline of the girder to the design section is computed in accordance with **LRFD [4.6.2.1.6]**. For steel beams, this distance is equal to one-quarter of the flange width from the centerline of support. For prestressed concrete girders, this distance is equal to the values presented in Figure 17.5-1, along with bar locations and clearances.

Note: Transverse reinforcing steel requirements (bar size and spacing) are determined for both positive moment requirements and negative moment requirements, and the same reinforcing steel is used in both the top and bottom of slab as shown in Table 17.5-1 and Table 17.5-2. Longitudinal reinforcement in Table 17.5-3 and Table 17.5-4 is based on a percentage of the bottom transverse reinforcement required *by actual design calculations* (not a percentage of what is in the tables). **The tables should be used for deck reinforcement, with continuity bars in prestressed girder bridges being the only deck reinforcement requiring calculation.**



Prestressed Girder	∆ (Inches)
28"	6
36"	4
36W", 45W"	11
54W", 72W", 82W"	15
45"	5
54"	6
70"	10

Figure 17.5-1
Transverse Section thru Slab on Girders

For skews of 20° and under, place transverse bars along the skew. For skews greater than 20°, place transverse bars perpendicular to the girders.

Detail "A", as presented in Figure 17.5-1, should be used for decks when shear connectors extend less than 2 inches into the slab on steel girder bridges or 3 inches on prestressed concrete girder bridges.

Several transverse reinforcing steel tables are provided in this chapter. The reinforcing steel in Table 17.5-1 and Table 17.5-2 does not account for deck overhangs. However, the minimum amount of reinforcing steel required in the deck overhangs is presented in various design tables in 17.6.

The reinforcement shown in Table 17.5-1 and Table 17.5-2 is based on both the Strength I requirement and crack control requirement.

Crack control was checked in accordance with **LRFD [5.7.3.4]**. The bar spacing cannot exceed the value from the following formula:

$$s \le \frac{700(\gamma)}{\beta_s f_s} - 2d_c$$

Where:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

γ	=	0.75 for decks
β_s	=	Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face
f_s	=	Tensile stress in reinforcement at the service limit state (ksi) ≤ 0.6 f_y
d_c	=	Top concrete cover less ½ inch wearing surface plus ½ bar diameter or bottom concrete cover plus ½ bar diameter (inches)
h	=	Slab depth minus ½ inch wearing surface (inches)

WisDOT policy item:

The thickness of the sacrificial ½-inch wearing surface shall not be included in the calculation of d_c.

Table 17.5-1 and Table 17.5-2 were developed for specified values of the distance from the centerline of girder to the design section for negative moment. Those specified values – 0, 3, 6, 9, 12 and 18 inches – were selected to match values used in **AASHTO [Table A4-1]**. For a girder in which the distance from the centerline of girder to the design section for negative

Effective Overhang (Feet)	Deck Thickness Between Girders, "t" (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.75	0.239	0.239	0.239	0.220	0.183	0.150	0.120
2.00	0.246	0.246	0.246	0.229	0.196	0.166	0.138
2.25	0.252	0.252	0.252	0.237	0.206	0.180	0.154
2.50	0.257	0.257	0.257	0.242	0.216	0.190	0.166
2.75	0.268	0.268	0.268	0.260	0.244	0.230	0.218
3.00	0.268	0.268	0.268	0.256	0.234	0.215	0.198
3.25	0.356	0.356	0.356	0.351	0.323	0.299	0.277
3.50	0.409	0.409	0.409	0.386	0.383	0.377	0.351
3.75	0.488	0.488	0.488	0.463	0.418	0.402	0.417
4.00	0.477	0.477	0.477	0.448	0.397	0.357	0.336

Table 17.6-4

Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Tubular Railing Type "M"
Girder Type 1

Effective Overhang (Feet)	Deck Thickness Between Girders, "t" (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.25	0.525	0.423	0.341	0.272	0.213	0.161	0.117
1.5	0.525	0.423	0.341	0.272	0.213	0.161	0.117
1.75	0.525	0.423	0.341	0.272	0.213	0.161	0.117
2	0.360	0.300	0.244	0.197	0.155	0.119	0.089
2.25	0.290	0.280	0.228	0.185	0.146	0.114	0.128
2.5	0.237	0.237	0.217	0.176	0.151	0.146	0.160
2.75	0.275	0.275	0.275	0.263	0.247	0.234	0.222
3	0.259	0.259	0.259	0.259	0.238	0.219	0.202
3.25	0.334	0.334	0.334	0.334	0.327	0.303	0.282

Table 17.6-5

Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Tubular Railing Type "M"
Girder Type 2

Notes:

1. Tables show the total area of transverse deck reinforcement required per foot.

2. The values in Table 17.6-2, Table 17.6-3, Table 17.6-4 and Table 17.6-5 are based on the following design criteria:

   - f'c = 4 ksi
   - fy = 60 ksi
   - Top steel clearance = 2 1/2"
   - Effective Overhang as illustrated in Figure 17.6-1

3. For Tubular Railing Type "M", the No. 6 "U" bars located at the rail post locations should not be included when calculating the total available area of reinforcement.

4. The values in the shaded region are satisfied by the standard transverse reinforcement for all girder spacings and standard transverse deck reinforcement. No additional checks or reinforcement are required.

5. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail "A" shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail "B" shall be used with Table 17.6-4.

6. For bridge decks with raised sidewalks, the additional reinforcement shown in Table 17.6-2, Table 17.6-3, Table 17.6-4, and Table 17.6-5, need not be used. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for information pertaining to the additional reinforcement to be used at raised sidewalks.

Example Use of Tables:

Given Information:

54W" PSG, 15" from CL girder to Design Section

Girder Spacing = 7'-0"

Overhang = 3'-0", Effective Overhang = 1'-9"

Type M rail

From Table 17.5-1:

Deck thickness = 8"

Design Section at 15", use #5's @ 8.5", As provided = 0.43 in^2/ft

From Table 17.6-5:

Transverse area of steel required = 0.525 in^2/ft

Therefore:

Additional area of steel required = 0.525-0.43 = 0.095 in^2/ft

Use either one or two times the spacing of the standard transverse reinforcement.

Lapping every other bar: use #4's @ 17", As = 0.14 in^2/ft, use Detail "A".

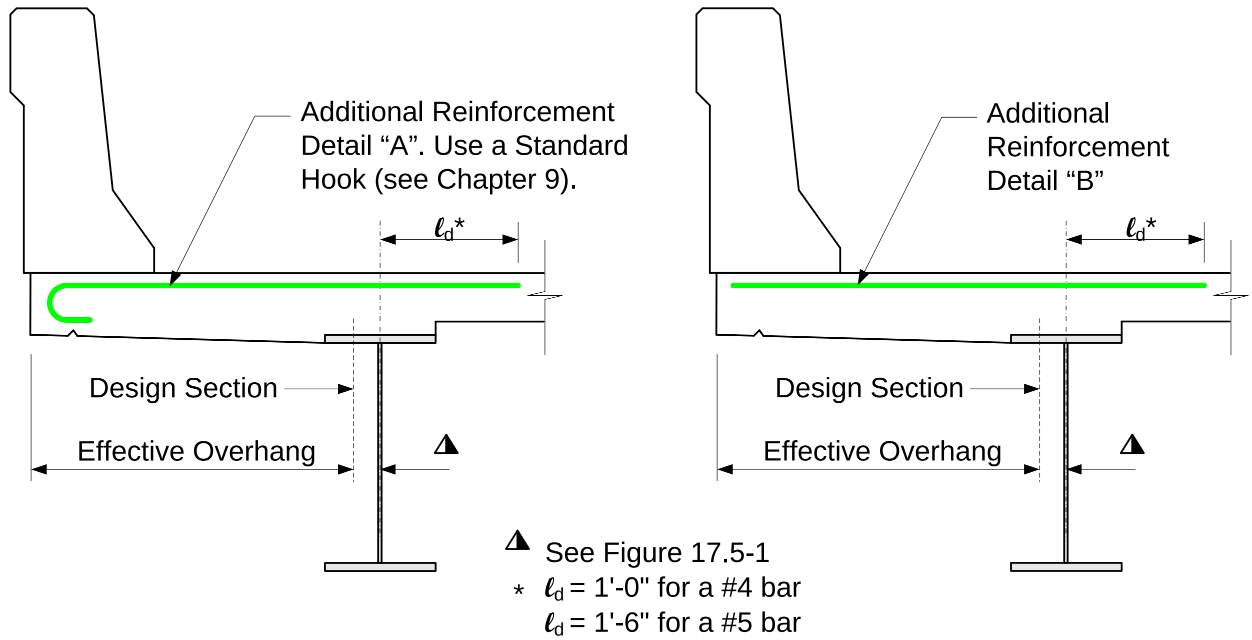


Figure 17.6-8
Overhang Reinforcement Details

To reiterate:

1. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail "A" shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail "B" shall be used with Table 17.6-4.

2. Girder Type 1 shall include steel girders and the following prestressed girders; 28-inch, 36-inch, 36W-inch, 45W-inch. Prestressed girders used for rehabilitation projects, 45-inch, 54-inch and 70-inch, shall also be considered to be Girder Type 1. Girder Type 2 shall include the following prestressed girders; 54W-inch, 72W-inch and 82W-inch.

17.7 Construction Joints

Optional transverse construction joints are permitted on continuous concrete deck structures to limit the concrete volume in a single pour. Refer to the Standard Detail for Slab Pouring Sequence for the optimum slab pouring sequence. On structures over 300 feet long, transverse construction joints, if used, are to be placed at 0.6 of the span length beyond the pier in the direction of the pour. For continuous prestressed concrete girder bridges, optional transverse construction joints should be located midway between the cut-off points for continuity reinforcing steel or at 0.75 of the span, whichever is closest to the pier.

The rate of placing concrete for continuous steel girders shall equal or exceed 0.5 of the span length per hour but need not exceed 100 cubic yards per hour. Transverse construction joints may be omitted with approval of Bureau of Structures.

Optional longitudinal construction joints, if used, are to be approved by the engineer and preferably located beneath the median or parapet. Otherwise, the joint should be located along the edge of the lane line. When the width of a superstructure exceeds 90 feet, a longitudinal construction joint with reinforcement through the joint shall be detailed. Longitudinal joints should also be at least 6 inches from the edge of the top flange of the girder. Open joints may be used in a median or between parapets. Consideration should be given to sealing open joints with compression seals or other sealants. A longitudinal construction joint detail is provided in the Standard Details.

The structure plans should permit the contractor to propose an alternate construction joint schedule subject to approval of the engineer.

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- 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 0.85 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 = β_1 (c) from the extreme compression fiber. The distance (c) is measured perpendicular to the neutral axis. 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]**.

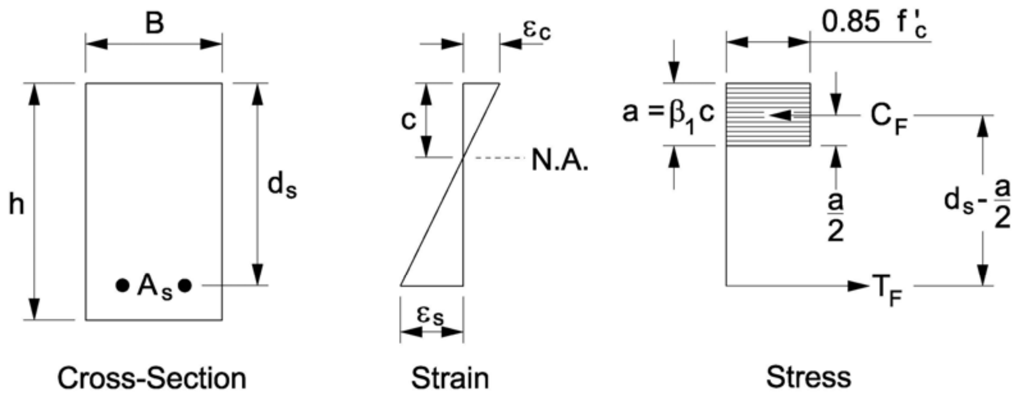

Figure 18.3-1
Stress / Strain on Cross - Section

Referring to Figure 18.3-1, the internal force equations are:

C_F = 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 / β_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:

$T_{cap} = (0.9)\ A_s\ f_y$

18.3.4 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**. The total factored force effect, Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

Service I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for crack control criteria
- Checking transverse slab reinforcement over the piers for crack control criteria
- Checking live load deflection criteria
- Checking dead load deflection (camber) criteria

18.3.4.1 Factored Loads

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Service I Limit State will be used to analyze the structure for force effects, Q_i , due to applied dead loads, DC and DW (including future wearing surface), defined in 18.4.2 and/or appropriate (HL-93) live loads, LL and IM, defined in 18.4.3.1. When sidewalks are present, include force effects of pedestrian live load, PL, where applicable, defined in 18.4.3.2.

For Service I Limit State, the values of γ_i for each applied load, are found in **LRFD [Table 3.4.1-1]** and their values are: $\gamma_{DC} = \gamma_{DW} = \gamma_{LL+IM} = \gamma_{PL} = 1.0$

Therefore, for Service I Limit State:

$$Q = 1.0\ [\ 1.0(DC) + 1.0(DW) + 1.0((LL + IM) + PL)\ ]$$

Where DC, DW, LL, IM, and PL represent force effects due to these applied loads.

18.3.4.2 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

The factored resistance, R_r , associated with the list of items to be checked using Service I Limit State in 18.3.4, are described in the following sections.

18.3.4.2.1 Crack Control Criteria

All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete.

Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

18.3.4.2.2 Live Load Deflection Criteria

All concrete slab structures shall be designed to meet live load deflection limits. The Bureau of Structures limits live load deflections for concrete slab structures to L/1200. The deflections are based on entire slab width acting as a unit and gross moment of inertia, I_g .

The nominal resistance, R_n , or deflection limit, is:

$$R_n = L/1200$$

Where:

L = span length

The factored resistance, R_r , is:

$$R_r = \phi R_n = \phi (L/1200)$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0) R_n = (L/1200)$$

18.3.4.2.3 Dead Load Deflection (Camber) Criteria

All concrete slab structures shall be designed to meet dead load deflection (camber) limits. Dead load deflections for concrete slab structures are computed using the gross moment of inertia, I_g . Bureau of Structures calculates full camber based on multiplying the dead load deflection values by a factor of three. A maximum allowable camber has been set for simple-span slabs and continuous-span slabs as shown in 18.4.4.2.

The nominal resistance, R_n , or deflection limit, is:

$$R_n = (\text{maximum allowable camber}) / 3$$

The factored resistance, R_r , is:

$$R_r = \phi R_n = \phi (\text{maximum allowable camber}) / 3$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0) R_n = (\text{maximum allowable camber}) / 3$$

18.3.5 Fatigue Limit State

Fatigue I Limit State shall be applied as a restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles **LRFD [1.3.2.3]**. The Fatigue I Limit State is intended to limit crack growth under repetitive loads to prevent fracture of the reinforcement during the design life of the bridge. The factored force effect (stress range), Q, must not exceed the factored resistance, R_r , as shown in the equation in 18.3.2.1.

For fatigue considerations, concrete members shall satisfy: **LRFD [5.5.3.1]**

$$\eta_i \, \gamma_i \, (\Delta f) \leq (\Delta F)_{TH}$$

Where:

γ_i	=	Load factor for Fatigue I Limit State
Δf	=	Force effect, live load stress range due to the passage of the fatigue truck (ksi)
$(\Delta F)_{TH}$	=	Constant-amplitude fatigue threshold (ksi)

Fatigue I Limit State **LRFD [3.4.1]** will be used for:

- Checking longitudinal slab reinforcement for fatigue stress range criteria

18.3.5.1 Factored Loads (Stress Range)

The value of the load modifier, η_i , is 1.00, as stated in 18.3.2.2.

Fatigue I Limit State will be used to analyze the structure for force effects, Q_i = (Δf), due to applied (Fatigue Truck) live load, LL and IM, defined in 18.4.3.1.

For Fatigue I Limit State, the value of γ_i for the applied live load, is found in **LRFD [Table 3.4.1-1]** and its value is $\gamma_{LL+IM} = 1.5$.

Therefore, for Fatigue I Limit State:

$$Q = 1.0 \, [\, 1.5(LL + IM) \,]$$

Where LL and IM represent force effects, Δf, due to these applied loads.

18.3.5.2 Factored Resistance

The resistance factor, ϕ, for Fatigue Limit State, is found in **LRFD [C1.3.2.1]** and its value is 1.00 .

18.3.5.2.1 Fatigue Stress Range

The nominal resistance, $R_n = (\Delta F)_{TH}$, for fatigue stress range (for straight reinforcement), is: **LRFD [5.5.3.2]**

$$R_n = (\Delta F)_{TH} = 24 - 20\, f_{min} / f_y \quad \text{(ksi)}$$

Where:

f_{min} = the minimum stress resulting from the factored Fatigue Truck live load, combined with the stress from the dead loads on the structure; positive if tension, negative if compression (ksi)

f_y = minimum yield strength (ksi), not to be taken less than 60 ksi nor greater than 100 ksi

The factored resistance, R_r (for f_y = 60 ksi), is:

$$R_r = \phi\, R_n = \phi\, (24 - 0.33\, f_{min})$$

The resistance factor, ϕ, is 1.00, therefore:

$$R_r = (1.0)\, R_n = 24 - 0.33\, f_{min} \quad \text{(ksi)}$$

Where:

E = equivalent distribution width (ft)

Look at the distribution factor (for axle loads) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

Compute the distribution factor associated with tributary portion of design lane load, to be applied to full lane load: **LRFD [3.6.1.2.4]**

$$DF = \frac{\left[\dfrac{(\text{SWL})}{(10\text{ ft lane load width})}\right]}{(\text{E})}$$

Where:

E = equivalent distribution width (ft)

SWL = Slab Width Loaded (with lane load) (ft) $\geq$ 0.

E – (distance from edge of slab to inside face of barrier) or

E – (distance from edge of slab to inside face of raised sidewalk)

Look at the distribution factor (for lane load) calculated for each span and select the largest value. This single value is to be applied along the entire length of the bridge.

18.4.6 Longitudinal Slab Reinforcement

The concrete cover on the top bars is 2 ½ inches, which includes a ½ inch wearing surface. The bottom bar cover is 1 ½ inches. Minimum clear spacing between adjacent longitudinal bars is 3 ½ inches. The maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the slab or 18.0 inches **LRFD [5.10.3.2]**. When bundled bars are used, see **LRFD [5.10.3.1.5, 5.11.2.3, 5.11.5.2.1]**.

18.4.6.1 Design for Strength

Strength Limit State considerations and assumptions are detailed in **LRFD [5.5.4, 5.7.2]**.

The area of longitudinal slab reinforcement, A_s , should be designed for strength at maximum moment locations along the structure, and for haunched slab structures, checked for strength at the haunch/slab intercepts. The area should also be checked for strength at bar reinforcement cutoff locations. This reinforcement should be designed for interior and exterior strips (edge beams) in both positive and negative moment regions. The reinforcement in the exterior strip is always equal to or greater than that required for the slab in an interior strip. Compare the reinforcement to be used for each exterior strip and select the strip with the largest amount of reinforcement (in^2/ft). Use this reinforcement pattern for both exterior strips to keep the bar layout symmetrical. Concrete parapets, curbs, sidewalks and other appurtenances are not to be considered to provide strength to the edge beam **LRFD [9.5.1]**. The total factored moment, M_u , shall be calculated using factored loads described in 18.3.3.1 for Strength I Limit State. Then calculate the coefficient of resistance, R_u :

$$R_u = M_u / \phi \, b \, d_s^2$$

Where:

ϕ	=	0.90 (see 18.3.3.2)
b	=	12 in (for a 1 foot design slab width)
d_s	=	slab depth (excl. ½ inch wearing surface) – bar clearance – ½ bar diameter (in)

Calculate the reinforcement ratio, ρ, using (R_u vs. ρ) Table 18.4-3 .

Then calculate required area,

$$A_s = \rho \, (b) \, (d_s)$$

Area of bar reinforcement per foot of slab width can be found in Table 18.4-4 .

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_u \leq M_r$ is satisfied.

The area of longitudinal reinforcement, A_s , should also be checked for moment capacity (factored resistance) along the structure, to make sure it can handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1. See Chapter 45 for details on checking the capacity of the structure for this Permit Vehicle.

18.4.6.2 Check for Fatigue

Fatigue Limit State considerations and assumptions are detailed in **LRFD [5.5.3, 5.7.1, 9.5.3]**

The area of longitudinal slab reinforcement, A_s , should be checked for fatigue stress range at locations where maximum stress range occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for fatigue stress range at bar reinforcement cutoff locations using Fatigue I Limit State. Check the reinforcement in an interior strip, where the largest number of fatigue cycles will occur.

Fatigue life of reinforcement is reduced by increasing the maximum stress level, bending of the bars and splicing of reinforcing bars by welding.

In regions where stress reversal takes place, continuous concrete slabs will be doubly reinforced. At these locations, the full stress range in the reinforcing bars from tension to compression is considered.

In regions of compressive stress due to permanent loads, fatigue shall be considered only if this compressive stress is less than 1.5 times the maximum tensile live load stress from the fatigue truck. The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads, and 1.5 times the fatigue load is tensile and exceeds $0.095\ (f'_c)^{½}$.

The factored stress range, Q, shall be calculated using factored loads described in 18.3.5.1. The factored resistance, R_r , shall be calculated as in 18.3.5.2.1.

Then check that, Q (factored stress range) $\leq R_r$ is satisfied.

Reference is made to the design example in 18.5 of this chapter for computations relating to reinforcement remaining in tension throughout the fatigue cycle, or going through tensile and compressive stresses during the fatigue cycle.

18.4.6.3 Check for Crack Control

Service Limit State considerations and assumptions are detailed in **LRFD [5.5.2, 5.7.1, 5.7.3.4]**.

The area of longitudinal slab reinforcement, A_s , should be checked for crack control at locations where maximum tensile stress occurs along the structure, and for haunched slab structures, checked at the haunch/slab intercepts. The area should also be checked for crack control at bar reinforcement cutoff locations using Service I Limit State. Check the reinforcement in an interior and exterior strip (edge beam).

The use of high-strength steels and the acceptance of design methods where the reinforcement is stressed to higher proportions of the yield strength, makes control of flexural cracking by proper reinforcing details more significant than in the past. The width of flexural cracks is proportional to the level of steel tensile stress, thickness of concrete cover over the bars, and spacing of reinforcement. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r , specified in **LRFD [5.4.2.6]**, for Service I Limit State. The spacing of reinforcement, s, in the layer closest to the tension face shall satisfy:

$$s \leq (700\ \gamma_e / \beta_s f_{ss}) - 2\ (d_c) \quad \text{(in)}$$

in which:

$$\beta_s = 1 + (d_c) / 0.7\ (h - d_c)$$

Where:

γ_e = 1.00 for Class 1 exposure condition (bottom reinforcement)

γ_e = 0.75 for Class 2 exposure condition (top reinforcement)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in)

f_{ss} = tensile stress in steel reinforcement (ksi) $\leq 0.6f_y$; use factored loads described in 18.3.4.1 at the Service I Limit State, to calculate (f_{ss})

h = overall depth of the section (in)

18.4.6.4 Minimum Reinforcement Check

The area of longitudinal slab reinforcement, A_s , should be checked for minimum reinforcement requirement at locations along the structure **LRFD [5.7.3.3.2]**.

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , or moment capacity, at least equal to the lesser of:

M_{cr} (or) 1.33 M_u

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

f_r = $0.37 (f'c)^{1/2}$ modulus of rupture (ksi) **LRFD [5.4.2.6]**

γ_1 = 1.6 flexural cracking variability factor

γ_3 = 0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement

I_g = gross moment of Inertia (in^4)

c = effective slab thickness/2 (in)

M_u = total factored moment, calculated using factored loads described in 18.3.3.1 for Strength I Limit State

Select lowest value of [M_{cr} (or) 1.33 M_u] = M_L

The factored resistance, M_r , or moment capacity, shall be calculated as in 18.3.3.2.1.

Then check that, $M_L \leq M_r$ is satisfied.

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 Sixth Edition - 2013 Interim***)

E18-1.1 Structure Preliminary Data

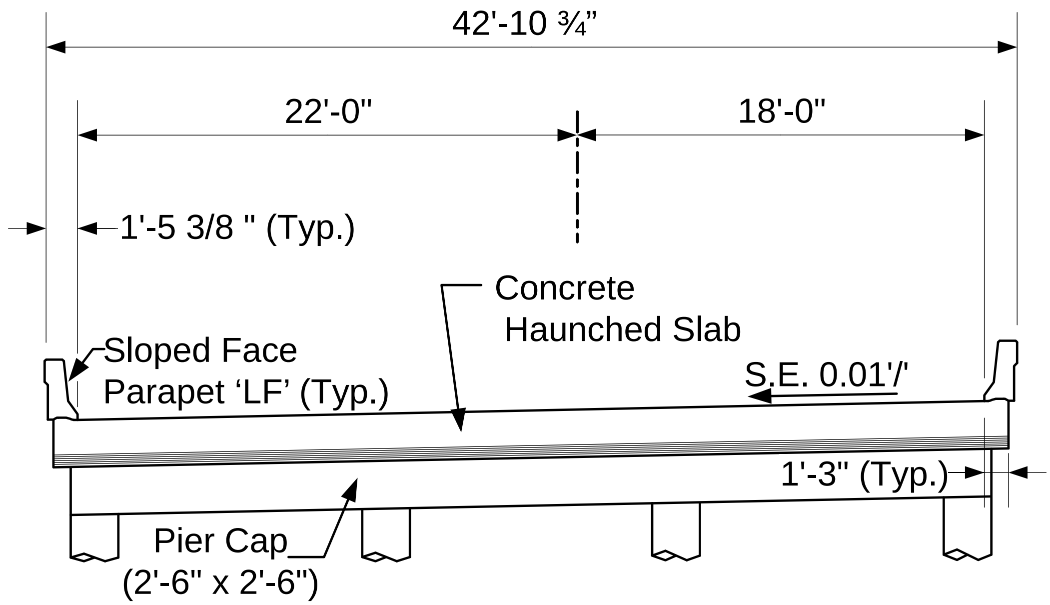

Figure E18.1

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)

$$Q = \Sigma \eta_i \cdot \gamma_i \cdot Q_i \leq \phi \cdot R_n = R_r \qquad \text{(Limit States Equation)}$$

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

$$Q = M_u = \eta_i\,[\gamma_{DCmax}(M_{DC}) + \gamma_{DWmax}(M_{DW}) + \gamma_{LLstr1}(M_{LL+IM})]$$

$$= 1.0\,[1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM})]$$

$$R_n = M_n = A_S \cdot f_S \cdot \left(d_S - \frac{a}{2}\right) \qquad \text{(See 18.3.3.2.1)}$$

$$M_r = \phi_f \cdot M_n = 0.90 \cdot A_S \cdot f_S \cdot \left(d_S - \frac{a}{2}\right)$$

Therefore : $M_u \le M_r$ (Limit States Equation)

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \le 0.90\, A_s\, f_s\, (d_s - a/2)$$

The positive live load moment shall be the largest caused by live loads (LL#1 or LL#2). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM)

From Table E18.4, the largest live load moment is from (LL#1), therefore at (0.4 pt. - span 1):

M_{DC} = 18.1 kip-ft M_{DW} = 1.5 kip-ft M_{LL+IM} = 7.9 + 37.5 = 45.4 kip-ft

$M_u := 1.25\cdot(18.1) + 1.50\cdot(1.5) + 1.75\cdot(45.4)$ $\boxed{M_u = 104.3}$ kip-ft

$b := 12$ inches (for a one foot design width)

$d_s = d_{slab}$ - bott. bar clr. - 1/2 bott. bar dia.

$d_s := 17 - 1.5 - 0.6$ $\boxed{d_s = 14.9}$ in

Calculate R_u, coefficient of resistance:

$$R_u = \frac{M_u}{\phi_f \cdot b \cdot d_s^2} \qquad R_u := \frac{104.3\cdot(12)\cdot 1000}{0.9\cdot(12)\cdot 14.9^2} \qquad \boxed{R_u = 522} \text{ psi}$$

Solve for ρ, reinforcement ratio, using Table 18.4-3 (R_u vs ρ) in 18.4.13;

$\rho := 0.0095$

$$A_s = \rho\cdot(b)\cdot d_s \qquad A_s := 0.0095\cdot(12)14.9 \qquad \boxed{A_s = 1.7}\ \frac{in^2}{ft}$$

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)

$$a = \frac{A_s\cdot f_y}{0.85\cdot f'_c\cdot b} \qquad a := \frac{1.71\cdot(60)}{0.85\cdot(4.0)\cdot 12} \qquad \boxed{a = 2.51} \text{ in}$$

If $\frac{c}{d_s} \le 0.6$ for (f_y= 60 ksi) **LRFD [5.7.2.1]**, then reinforcement has yielded and the assumption is correct.

$$\beta_1 := 0.85 \qquad c := \frac{a}{\beta_1} \qquad \boxed{c = 2.96} \text{ in}$$

$\frac{c}{d_s} = 0.2 < 0.6$ therefore, the reinforcement will yield.

$$M_r = 0.90\cdot A_s\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$$

$$M_r := 0.9\cdot(1.71)\cdot 60.0\cdot\left(\frac{14.9 - \frac{2.51}{2}}{12}\right) \qquad \boxed{M_r = 105} \text{ kip-ft}$$

Therefore, M_u = 104.3 kip-ft < M_r = 105 kip-ft O.K.

E18-1.7.1.2 Check for Fatigue

Check reinforcement using Fatigue I Limit State and considerations and assumptions detailed in **LRFD [5.5.3, 5.7.1, 9.5.3]**.

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLfatigue} := 1.5$ $\phi_{fatigue} := 1.0$

When reinforcement remains in tension throughout the fatigue cycle,

$Q_i = \Delta f = f_{range}$ = stress range in bar reinforcement due to flexural moment range (M_{range}) caused by Fatigue Truck (LL#4). See Table E18.2 and E18.3 in E18-1.4 for description of live load and dynamic load allowance (IM)

$$Q = \eta_i \cdot \gamma_{LLfatigue} \cdot f_{range} = (1.0) \cdot (1.5) \cdot f_{range}$$

$$R_n = (\Delta F_{TH}) = 24 - 0.33 \cdot f_{min}$$ for f_y = 60 ksi (See 18.3.5.2.1)

$$R_r = \phi_{fatigue} \cdot R_n = 1.0 \cdot (24 - 0.33 \cdot f_{min})$$

Therefore: $1.5 \cdot (f_{range}) \le 24 - 0.33 \cdot f_{min}$ (Limit States Equation)

From Table E18.4, the moments at (0.4 pt.) of span 1 are:

M_{DC} = 18.1 kip-ft M_{DW} = 1.5 kip-ft

+Fatigue Truck = 16.7 kip-ft -Fatigue Truck = -5.5 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

The section properties for fatigue shall be based on cracked sections where the sum of stresses, due to permanent loads, and ($\gamma_{LLfatigue}$ = 1.5) times the fatigue load is tensile and exceeds $0.095\sqrt{f'_c}$ **LRFD [5.5.3.1]**

Allowable tensile stress for fatigue (cracking stress):

$$f_{tensile} = 0.095\sqrt{f'_c} = 0.095 \cdot \sqrt{4}$$ $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) +1.5(Fatigue Truck)

$M_{fatigueMax} := 1.0 \cdot (18.1) + 1.0(1.5) + 1.5(16.7)$ $\boxed{M_{fatigueMax} = 44.65}$ kip-ft (tension)

$M_{fatigueMin} := 1.0 \cdot (18.1) + 1.0(1.5) + 1.5(-5.5)$ $\boxed{M_{fatigueMin} = 11.35}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$: $f_{fatigue} = \dfrac{M_{fatigue} \cdot (y)}{I_g}$

$y = \dfrac{d_{slab}}{2} = \dfrac{17}{2}$ $\boxed{y = 8.5}$ in

$I_g = \dfrac{1}{12} \cdot b \cdot d_{slab}^{\,3} = \dfrac{1}{12} \cdot (12)\ 17^3$ $\boxed{I_g = 4913}$ in^4

$f_{fatigueMax} := \dfrac{M_{fatigueMax} \cdot (y) \cdot 12}{I_g}$ $\boxed{f_{fatigueMax} = 0.93}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$f_{fatigueMin} := \dfrac{M_{fatigueMin} \cdot (y) \cdot 12}{I_g}$ $\boxed{f_{fatigueMin} = 0.24}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (+ Fatigue Truck) - (-Fatigue Truck)

$M_{range} := 16.7 - (-5.5)$ $\boxed{M_{range} = 22.2}$ kip-ft

The moment arm used in equations below is: (j) (d_s) Therefore, using:

$A_s = 1.7 \ \dfrac{in^2}{ft}$ (required for strength), $d_s = 14.9$ in , $n := 8$, and transformed section analysis, gives a value of $j := 0.893$

$f_{range} = \dfrac{M_{range}}{A_s \cdot (j) \cdot d_s} = \dfrac{22.2 \cdot 12}{1.7 \cdot (0.893)\ 14.9}$ $\boxed{f_{range} = 11.78}$ ksi

$f_{range1.5} := 1.5 \cdot f_{range}$ $\boxed{f_{range1.5} = 17.67}$ ksi

$$f_{min} = \frac{M_{DC} + M_{DW} + 1.5(-FatigueTruck)}{A_s \cdot (j) \cdot d_s}$$

$f_{min} := \dfrac{[18.1 + 1.5 + 1.5 \cdot (-5.5)] \cdot 12}{1.7 \cdot (0.893)\ 14.9}$ $\boxed{f_{min} = 6.02}$ ksi

$R_r := 24 - 0.33 \cdot f_{min}$ $\boxed{R_r = 22.01}$ ksi

Therefore, $1.5 \cdot (f_{range}) = 17.67$ ksi < $R_r = 22.01$ ksi <u>O.K.</u>

E18-1.7.1.3 Check Crack Control

Check reinforcement using Service I Limit State and considerations and assumptions detailed in **LRFD [5.5.2, 5.7.1, 5.7.3.4]**

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

$f_r = 0.24 \cdot \sqrt{f'_c}$ $\quad \boxed{f_r = 0.48}$ ksi $\quad f_{r80\%} := 0.8 \cdot f_r$ $\quad \boxed{f_{r80\%} = 0.38}$ ksi

$$f_T = \frac{M_S \cdot (c)}{I_g}$$

$c := \frac{d_{slab}}{2}$ $\quad \boxed{c = 8.5}$ in $\quad I_g := \frac{1}{12} \cdot b \cdot d_{slab}^{\ 3}$ $\quad \boxed{I_g = 4913}$ in^4

Looking at E18-1.2: $\eta_i := 1.0$

and from Table E18.1: $\gamma_{DC.ser1} := 1.0$ $\quad \gamma_{DW.ser1} := 1.0$ $\quad \gamma_{LLser1} := 1.0$ $\quad \phi_{ser1} := 1.0$

$Q_i = M_{DC}, M_{DW}, M_{LL+IM}$ **LRFD [3.6.1.2, 3.6.1.3.3]**; moments due to applied loads as stated in E18-1.2

$Q = M_S = \eta_i \left[\gamma_{DC.ser1}(M_{DC}) + \gamma_{DW.ser1}(M_{DW}) + \gamma_{LLser1}(M_{LL+IM})\right]$

$= 1.0\left[1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})\right]$

Therefore, M_s becomes:

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$ (Factored Load Equation)

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.1.1, at (0.4 pt.) of span 1, provides:

$M_{DC} = 18.1$ kip-ft $\quad M_{DW} = 1.5$ kip-ft $\quad M_{LL+IM} = 7.9 + 37.5 = 45.4$ kip-ft (LL#1)

$M_S := 1.0 \cdot (18.1) + 1.0 \cdot (1.5) + 1.0 \cdot (45.4)$ $\quad \boxed{M_S = 65}$ kip-ft

$f_T = \frac{M_S \cdot (c)}{I_g}$ $\quad f_T := \frac{65.0 \cdot (8.5) \cdot 12}{4913}$ $\quad \boxed{f_T = 1.35}$ ksi

$f_T = 1.35$ ksi > 80% $f_r = 0.38$ ksi; therefore, check crack control criteria

Knowing $A_S = 1.7 \frac{in^2}{ft}$ (required for strength)

Try: #9 at 7" c-c spacing ($A_S = 1.71\ in^2/ft$) from Table 18.4-4 in 18.4.13

The spacing (s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot (d_c) \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

d_c = clr. cover + 1/2 bar dia.
= thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in). See Figure E18.3

$$d_c := 1.5 + \frac{1.128}{2} \qquad \boxed{d_c = 2.064} \text{ in}$$

h = overall depth of the section (in). See Figure E18.3

$$h := d_{slab} \qquad \boxed{h = 17} \text{ in}$$

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.2}$$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\le 0.6\ f_y$

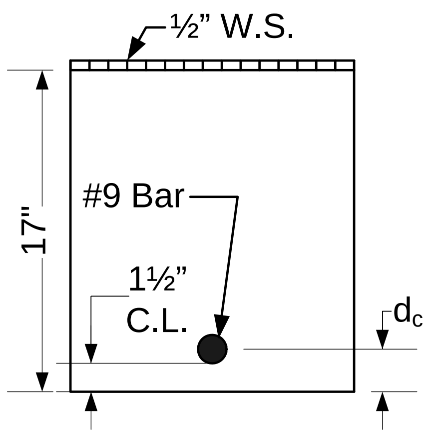

Figure E18.3

Cross Section - (0.4 pt.) Span 1

The moment arm used in the equation below to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.1.2, j = 0.893

$$f_{ss} = \frac{M_s}{A_s \cdot (j) \cdot (h - d_c)} = \frac{65.0 \cdot (12)}{1.71 \cdot (0.893)(17 - 2.064)} \qquad \boxed{f_{ss} = 34.2} \text{ ksi} \quad \le 0.6\ f_y \text{ O.K.}$$

$$s \le \frac{700 \cdot (1.00)}{1.2 \cdot (34.2)} - 2 \cdot (2.064) = 17.0 - 4.1 = 12.9 \quad \text{in}$$

$s \le 12.9$ in

Therefore, spacing prov'd. = 7 in < 12.9 in <u>O.K.</u>

<u>Use: #9 at 7" c-c spacing in span 1</u> (Max. positive reinforcement).

E18-1.7.1.4 Minimum Reinforcement Check

The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance (M_r), or moment capacity, at least equal to the lesser of: **LRFD[5.7.3.3.2]**

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r) S$ where: $S = \frac{I_g}{c}$ therefore, $M_{cr} = 1.1(f_r)\frac{I_g}{c}$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement <u>for A615, Grade 60 reinforcement</u>

$f_r = 0.37\sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r = 0.37\sqrt{4}$ $\boxed{f_r = 0.74}$ ksi

$I_g := \frac{1}{12} \cdot b \cdot d_{slab}{}^3$ $\boxed{I_g = 4913}$ in^4 $c := \frac{d_{slab}}{2}$ $\boxed{c = 8.5}$ in

$$M_{cr} = \frac{1.1f_r \cdot (I_g)}{c} = \frac{1.1 \cdot 0.74 \cdot (4913)}{8.5(12)}$$ $\boxed{M_{cr} = 39.21}$ kip-ft

$1.33 \cdot M_u = 138.75$ kip-ft , where M_u was calculated for Strength Design in E18-1.7.1.1 and (M_u = 104.3 kip-ft)

M_{cr} controls because it is less than 1.33 M_u

As shown in E18-1.7.1.1, the reinforcement yields, therefore:

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$ $\boxed{M_r = 105}$ kip-ft

Therefore, M_{cr} = 39.21 kip-ft < M_r = 105 kip-ft <u>O.K.</u>

E18-1.7.2 Negative Moment Reinforcement at Piers

Examine at C/L of Pier

E18-1.7.2.1 Design for Strength

Following the procedure in E18-1.7.1.1, using Strength I Limit State:

$$M_u = 1.25(M_{DC}) + 1.50(M_{DW}) + 1.75(M_{LL+IM}) \leq 0.90\, A_s\, f_s\, (d_s - a/2)$$

The negative live load moment shall be the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM).

From Table E18.4, the largest live load moment is from (LL#2), therefore at (C/L of Pier):

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft

$M_u := 1.25\cdot(-59.2) + 1.50\cdot(-4.9) + 1.75\cdot(-55.4)$ $\boxed{M_u = -178.3}$ kip-ft

$b := 12$ inches (for a one foot design width) and $\boxed{d_s = 25.4}$ in

The coefficient of resistance, R_u, the reinforcement ratio, ρ, and req'd. bar steel area, A_s, are:

$\boxed{R_u = 307.1}$ psi $\boxed{\rho = 0.0054}$ $\boxed{A_s = 1.65}$ $\frac{in^2}{ft}$

Try: #8 at 5 1/2" c-c spacing (A_s = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

Assume $f_s = f_y$, then the depth of the compressive stress block is: $\boxed{a = 2.51}$ in

Then, $\boxed{c = 2.96}$ in and $\frac{c}{d_s} = 0.12 < 0.6$ therefore, the reinforcement will yield.

The factored resistance is: $\boxed{M_r = 186.6}$ kip-ft

Therefore, M_u = 178.3 kip-ft < M_r = 186.6 kip-ft   O.K.

E18-1.7.2.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.5\cdot\left(f_{range}\right) \leq 24 - 0.33\cdot f_{min}$ (for f_y = 60 ksi)

From Table E18.4, the moments at (C/L Pier) are:

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft

+Fatigue Truck = 3.9 kip-ft -Fatigue Truck = -23.0 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

Allowable tensile stress for fatigue (cracking stress): $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) +1.5(\text{Fatigue Truck})$

$\boxed{M_{fatigueMax} = -98.6}$ kip-ft (tension) $\boxed{M_{fatigueMin} = -58.3}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$, where : $\boxed{y = 14}$ in $\boxed{I_g = 21952}$ in^4

$$f_{fatigue} = \frac{M_{fatigue} \cdot (y)}{I_g}$$

$\boxed{f_{fatigueMax} = 0.75}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$\boxed{f_{fatigueMin} = 0.45}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (- Fatigue Truck) - (+Fatigue Truck) $\boxed{M_{range} = -26.9}$ kip-ft

The values for A_s, d_s, n and j (from transformed section) used to calculate f_{range} and f_{min} are:

$A_S = 1.65 \frac{in^2}{ft}$ (required for strength), $d_S = 25.4$ in , $n := 8$, $j := 0.915$

The values for f_{range}, $f_{range1.5}$, and f_{min} are:

$\boxed{f_{range} = 8.42}$ ksi $\boxed{f_{range1.5} = 12.63}$ ksi $\boxed{f_{min} = 18.23}$ ksi

The factored resistance is: $\boxed{R_r = 17.98}$ ksi

Therefore, $1.5 \cdot (f_{range}) = 12.63$ ksi < $R_r = 17.98$ ksi O.K.

E18-1.7.2.3 Check Crack Control

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 14}$ in $\boxed{I_g = 21952}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.2.1, at (C/L of Pier), provides:

M_{DC} = -59.2 kip-ft M_{DW} = -4.9 kip-ft M_{LL+IM} = -15.5 + (-39.9) = -55.4 kip-ft (LL#2)

$M_S := 1.0 \cdot (59.2) + 1.0 \cdot (4.9) + 1.0 \cdot (55.4)$ $\boxed{M_S = 119.5}$ kip-ft

$f_T = \dfrac{M_S \cdot c}{I_g}$ $f_T := \dfrac{119.5 \cdot (14) \cdot 12}{21952}$ $\boxed{f_T = 0.91}$ ksi

f_T =0.91 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

Knowing $A_S = 1.65 \dfrac{in^2}{ft}$ (required for strength)

Try: #8 at 5 1/2" c-c spacing (A_S = 1.71 in^2/ft) from Table 18.4-4 in 18.4.13

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in (See Figure E18.4) $\boxed{h = 28}$ in (See Figure E18.4) $\boxed{\beta_S = 1.14}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{SS} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.2.2, j = 0.915

The value of f_{SS} and (s) are:

$\boxed{f_{SS} = 35.94}$ ksi $\leq 0.6\ f_y$ O.K. $s \leq \dfrac{700 \cdot (0.75)}{1.14 \cdot (35.94)} - 2 \cdot (2.50) = 12.8 - 5.0 = 7.8$ in

$s \leq 7.8$ in

Therefore, spacing prov'd. = 5 1/2 in < 7.8 in <u>O.K.</u>

To insure that the reinforcement has the moment capacity to handle the Wisconsin Standard Permit Vehicle (Wis-SPV), the spacing was reduced to 5 inches. (See E18-1.8)

<u>Use: #8 at 5" c-c spacing at C/L Pier</u> (Max. negative reinforcement), $A_S = 1.88 \dfrac{in^2}{ft}$

E18-1.7.3.2 Check for Fatigue

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.5 \cdot (f_{range}) \le 24 - 0.33 \cdot f_{min}$ (for f_y = 60 ksi)

From Table E18.4, the moments at (0.5 pt.) of span 2 are:

M_{DC} = 19.6 kip-ft M_{DW} = 1.6 kip-ft

+Fatigue Truck = 16.7 kip-ft   -Fatigue Truck = -3.4 kip-ft

In regions of tensile stress due to permanent loads, fatigue criteria should be checked.

Allowable tensile stress for fatigue (cracking stress): $\boxed{f_{tensile} = 0.19}$ ksi

Calculate fatigue moment and then select section properties:

$M_{fatigue}$ = 1.0(M_{DC}) + 1.0(M_{DW}) +1.5(Fatigue Truck)

$\boxed{M_{fatigueMax} = 46.25}$ kip-ft (tension) $\boxed{M_{fatigueMin} = 16.1}$ kip-ft (tension)

Calculate stress due to $M_{fatigue}$, where : $\boxed{y = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$$f_{fatigue} = \frac{M_{fatigue} \cdot (y)}{I_g}$$

$\boxed{f_{fatigueMax} = 0.96}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

$\boxed{f_{fatigueMin} = 0.33}$ ksi (tension) > $f_{tensile}$ (0.190 ksi)

Values of $f_{fatigue}$ exceed $f_{tensile}$ during the fatigue cycle, therefore analyze fatigue using cracked section properties.

Looking at values of $M_{fatigue}$, shows that the reinforcement remains in tension throughout the fatigue cycle. Therefore:

M_{range} = (+ Fatigue Truck) - (-Fatigue Truck) $\boxed{M_{range} = 20.1}$ kip-ft

The values for A_s, d_s, n and j (from transformed section) used to calculate f_{range} and f_{min} are:

$A_s = 1.73 \ \frac{in^2}{ft}$ (required for strength), $d_s = 14.9$ in , $n := 8$, $j := 0.892$

The values for f_{range}, $f_{range1.5}$, and f_{min} are:

$\boxed{f_{range} = 10.43}$ ksi $\boxed{f_{range1.5} = 15.64}$ ksi $\boxed{f_{min} = 8.4}$ ksi

The factored resistance is: $\boxed{R_r = 21.23}$ ksi

Therefore, $1.5 \cdot (f_{range})$ = 15.64 ksi < R_r = 21.23 ksi   O.K.

E18-1.7.3.3 Check Crack Control

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Using same moments selected from Table E18.4 for Strength Design in E18-1.7.3.1, at (0.5 pt.) of span 2 provides:

$M_{DC} = 19.6$ kip-ft $M_{DW} = 1.6$ kip-ft $M_{LL+IM} = 8.2 + 37.4 = 45.6$ kip-ft (LL#1)

$M_S := 1.0\cdot(19.6) + 1.0\cdot(1.6) + 1.0\cdot(45.6)$ $\boxed{M_S = 66.8}$ kip-ft

$f_T = \dfrac{M_S \cdot c}{I_g}$ $f_T := \dfrac{66.8\cdot(8.5)\cdot 12}{4913}$ $\boxed{f_T = 1.39}$ ksi

$f_T = 1.39$ ksi > 80% $f_r = 0.38$ ksi; therefore, check crack control criteria

Knowing $A_S = 1.73 \ \dfrac{in^2}{ft}$ (required for strength)

Try: #9 at 6" c-c spacing ($A_S = 2.00\ in^2/ft$) from Table 18.4-4 in 18.4.13

The values for γ_e , d_c , h , and β_S , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

$\boxed{d_c = 2.064}$ in (See Figure E18.5) $\boxed{h = 17}$ in (See Figure E18.5) $\boxed{\beta_S = 1.2}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{SS} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.3.2, j = 0.892

The value of f_{SS} and (s) are:

$\boxed{f_{SS} = 30.08}$ ksi $\leq 0.6\ f_y$ O.K. $s \leq \dfrac{700\cdot(1.00)}{1.2\cdot(30.08)} - 2\cdot(2.064) = 19.4 - 4.1 = 15.3$ in

$s \leq 15.3$ in

Therefore, spacing prov'd. = 6 in < 15.3 in <u>O.K.</u>

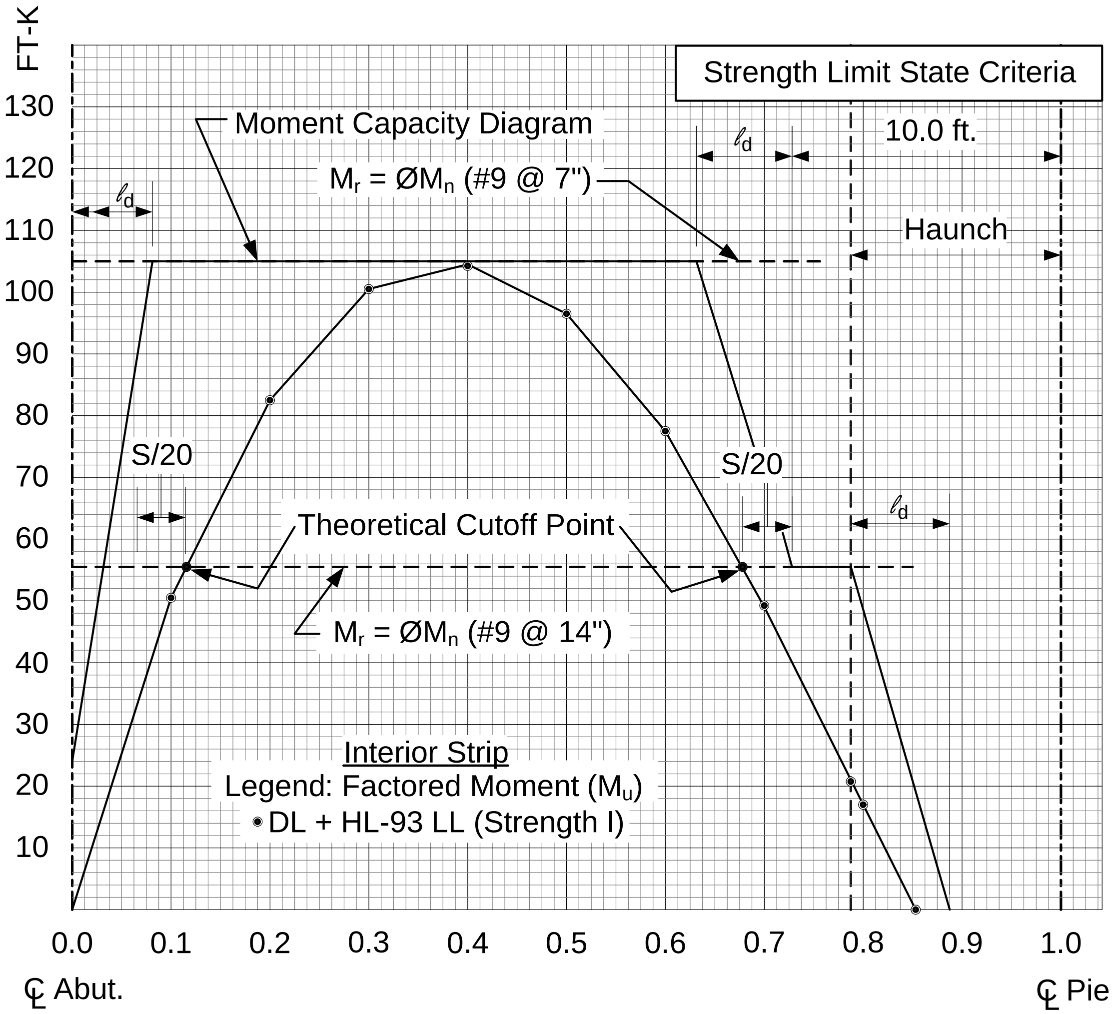

Figure E18.6

Span (1) - Positive Moment Cutoff Diagram

The moment diagram equals the capacity of #9 at 14" at 4.2 (ft) from the C/L of the abutment. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.11.1.2.1]**

$d_{eff} := 14.9$ in $\qquad$ ℓ_d (#9) (See Table 9.9-2, Chapter 9).

$15 \cdot (d_b) = 15 \cdot (1.128) = 16.9$ in

$\frac{S}{20} = \frac{38}{20} = 1.9$ ft $\qquad$ controls

Therefore, 1/2 of bars may be cut at 2.0 (ft) from the C/L of the abutment if fatigue and crack control criteria are satisfied.

Because the cutoff point is close to the abutment, don't cut 1/2 of bars, but run all #9 bars into the support. **LRFD [5.11.1.2.2]**

The moment diagram equals the capacity of #9 at 14" at 12.1 (ft) from the C/L of pier. Reinforcement shall be extended S/20 beyond this point.

Therefore, 1/2 of bars may be cut at 10.0 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.74 pt.)

E18-1.7.5.1.1 Fatigue Check (at Cutoff) - (0.74 Pt.)

Following the procedure in E18-1.7.1.2, using Fatigue I Limit State:

$1.5 \cdot (f_{range}) \leq 24 - 0.33 \cdot f_{min}$ $\qquad$ (for f_y = 60 ksi)

Interpolating from Table E18.4, the moments at (0.74 pt.) of span 1 are:

M_{DC} = -10.0 kip-ft $\qquad$ M_{DW} = -0.89 kip-ft

+Fatigue Truck = 9.72 kip-ft $\qquad$ -Fatigue Truck = -10.34 kip-ft

In regions of compressive stress due to permanent loads, fatigue shall be considered only if this compressive stress is less than ($\gamma_{LLfatigue}$ = 1.5) times the maximum tensile stress from the fatigue load. **LRFD [5.5.3.1]**

For simplicity, assume fatigue criteria should be checked.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0 \cdot (-10.0) + 1.0(-0.89) + 1.5(9.72)$ $\qquad$ $\boxed{M_{fatigueMax} = 3.69}$ kip-ft (tens.)

$M_{fatigueMin} := 1.0 \cdot (-10.0) + 1.0(-0.89) + 1.5(-10.34)$ $\qquad$ $\boxed{M_{fatigueMin} = -26.4}$ kip-ft (compr.)

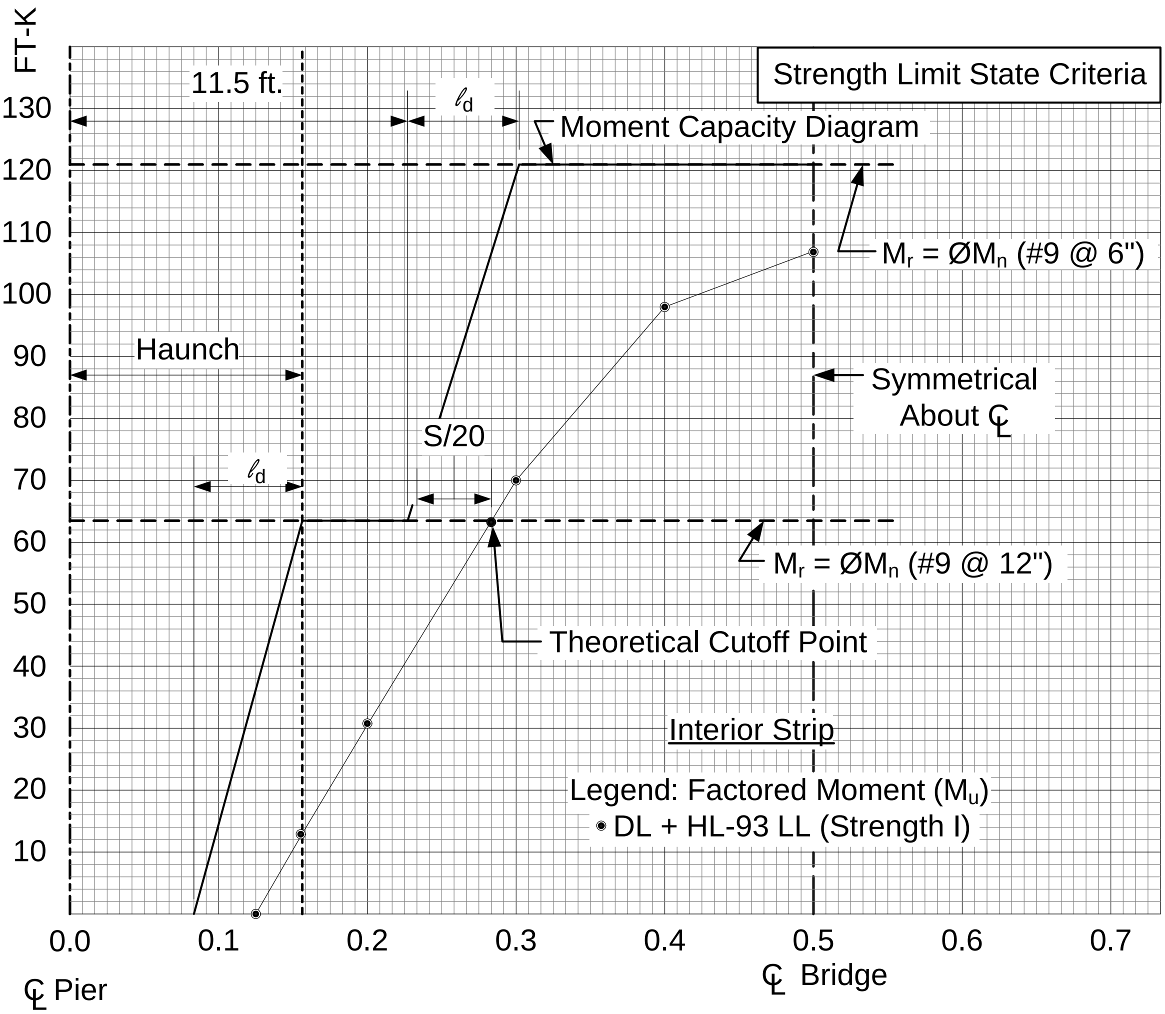

Figure E18.7

Span (2) - Positive Moment Cutoff Diagram

E18-1.7.5.2.1 Fatigue Check (at Cutoff) - (0.23 Pt.)

Looking at E18-1.2: $\eta_i := 1.0$ and from Table E18.1: $\gamma_{LLfatigue} := 1.5$ $\phi_{fatigue} := 1.0$

When reinforcement goes through tensile and compressive stress during the fatigue cycle,

$$Q = f_s + f'_s$$

Where:

f_s = tensile part of stress range in bar reinforcement due to dead load moments from applied loads in E18-1.2 and largest factored tensile moment caused by Fatigue Truck (LL#4)

f'_s = compressive part of stress range in bar reinforcement due to dead load moments from applied loads in E18-1.2 and largest factored compressive moment caused by Fatigue Truck (LL#4)

All live load moments in f_s and f'_s are multiplied by (η_i) and $(\gamma_{LLfatigue})$

See Table E18.2 and E18.3 in E18-1.4 for description of live load and dynamic load allowance (IM).

$$R_n = (\Delta F_{TH}) = 24 - 0.33 \cdot f_{min}$$ (for f_y = 60 ksi) (See 18.3.5.2.1)

$$R_r = \phi_{fatigue} \cdot R_n = 1.0 \cdot (24 - 0.33 \cdot f_{min})$$

Therefore: $f_s + f'_s \le 24 - 0.33 \cdot f_{min}$ (Limit States Equation)

Interpolating from Table E2, the moments at (0.23 pt.) of span 2 are:

M_{DC} = -3.5 kip-ft M_{DW} = -0.31 kip-ft

+Fatigue Truck = 10.02 kip-ft -Fatigue Truck = -7.3 kip-ft

In regions of compressive stress due to permanent loads, fatigue shall be considered only if this compressive stress is less than ($\gamma_{LLfatigue}$ = 1.5) times the maximum tensile stress from the fatigue load. **LRFD [5.5.3.1]**

The section properties for fatigue shall be based on cracked sections where the sum of stresses, due to permanent loads, and ($\gamma_{LLfatigue}$ = 1.5) times the fatigue load is tensile and exceeds $0.095\sqrt{f'_c}$.

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0(-3.5) + 1.5(10.02)$ $\boxed{M_{fatigueMax} = 11.53}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(-3.5) + 1.5(-7.3)$ $\boxed{M_{fatigueMin} = -14.45}$ kip-ft (compression)

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile range.

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.8, for definition of d_1, d_2, d', A_s and A'_s.

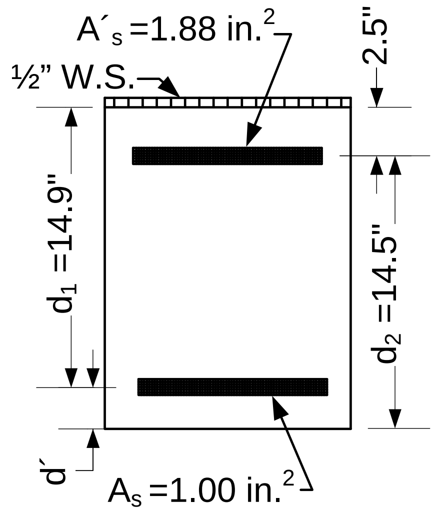

Figure E18.8

Cross Section - (0.23 pt.) Span 2

The moment arm used in equations below is: $(j_1)\,(d_1)$ for finding f_s
$(j_2)\,(d_2)$ for finding f'_s

Using: A_s = 1.00 in^2/ft, d_1 = 14.9 in, n = 8, and transformed section analysis, gives a value of $j_1 = 0.914$

Using: A'_s = 1.88 in^2/ft, d_2 = 14.5 in, n = 8, and transformed section analysis, gives a value of $j_2 = 0.887$; $k = x/d_2 = 0.34$, where x = distance from compression face to neutral axis

The tensile part of the stress range in the bottom bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1}$$ $\boxed{f_s = 10.16}$ ksi (tension)

The compressive part of the stress range in the bottom bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k}$$ $\boxed{f'_s = -2.15}$ ksi (compression)

It is assumed (#8's at 5") req'd. at pier, is present at this location as compression steel (A'_s).

Therefore, total stress range on bottom steel:

$f_s + f'_s = 10.16 - (-2.15) = 12.31$ ksi

$R_r := 24 - 0.33 \cdot f_{min}$ where $f_{min} = f'_s$, therefore: $\boxed{R_r = 24.71}$ ksi

Therefore, $f_s + f'_s = 12.31$ ksi < $R_r = 24.71$ ksi O.K..

E18-1.7.5.2.2 Crack Control Check (at Cutoff) - (0.23 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.23 pt.) of span 2 are:

M_{DC} = -3.51 kip-ft M_{DW} = -0.31 kip-ft M_{LL+IM} = 3.7 + (21.9) = 25.6 kip-ft (LL#1)

$M_s := 1.0 \cdot (-3.51) + 1.0 \cdot (25.6)$ $\boxed{M_s = 22.1}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment.

$f_T = \frac{M_s \cdot c}{I_g}$ $f_T := \frac{22.1 \cdot (8.5) \cdot 12}{4913}$ $\boxed{f_T = 0.46}$ ksi

f_T =0.46 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

For: #9 at 12" c-c spacing ($A_s = 1.00\ in^2/ft$)

The values for γ_e , d_c , h , and β_s , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 1.00$ for Class 1 exposure condition (bottom reinforcement)

$\boxed{d_c = 2.064}$ in $\boxed{h = 17}$ in $\boxed{\beta_s = 1.2}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)

As shown in fatigue calculations in E18-1.7.5.2.1, j = 0.914

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 19.43}$ ksi $\leq 0.6\, f_y$ O.K. $\quad s \leq \dfrac{700 \cdot (1.00)}{1.2 \cdot (19.43)} - 2 \cdot (2.064) = 30.07 - 4.1 = 26.0$ in

$s \leq 26.0$ in

Therefore, spacing prov'd. = 12 in < 26.0 in O.K.

E18-1.7.5.2.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.1.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 11.5 (ft) from the C/L of each pier. Remaining bars are extended (ℓ_d) beyond the haunch/slab intercept as shown on Standard 18.01.

E18-1.7.5.3 Span 1 Negative Moment Reinforcement (Cutoffs)

Theoretical bar steel cutoff points for negative moment are determined when one-half the steel required at the (C/L Pier) has the moment capacity, or factored resistance , M_r , equal to the total factored moment , M_u , at these points. However, the remaining bars are to be extended beyond the theoretical cutoff point and must meet the fatigue and crack control requirements at these cutoff locations. The factored moments, M_u , at the 0.1 points and haunch/slab intercepts have been plotted on Figure E18.9. The capacities, M_r , of #8 at 5" and #8 at 10" are also shown. The factored moments, M_u , and capacities, M_r , are based on Strength I Limit State criteria. The negative live load moments, M_{LL+IM} , used to calculate M_u are taken as the largest caused by live loads (LL#1, LL#2 or LL#3). See Table E18.2 and E18.3 in E18-1.4 for description of live loads and dynamic load allowance (IM). Use maximum or minimum load factors for M_{DC} and M_{DW} (See Table E18.1) to calculate the critical force effect. When value of M_{DW} is (+), assume FWS is not present and ignore it.

Calculate the capacity of #8 at 5" c-c spacing $\quad A_s := 1.88 \ \dfrac{in^2}{ft}$

$b = 12$ inches (for a one foot design width)

As shown in E18-1.7.2.1, reinforcement will yield, therefore: $\boxed{a = 2.76}$ in

$M_r = 204.1$ kip-ft (at C/L pier), $\quad d_s := 25.5$ in

$M_r = 111.0$ kip-ft (in span), $\quad d_s := 14.5$ in

Calculate the capacity of #8 at 10" c-c spacing $\quad A_s := 0.94 \ \dfrac{in^2}{ft}$

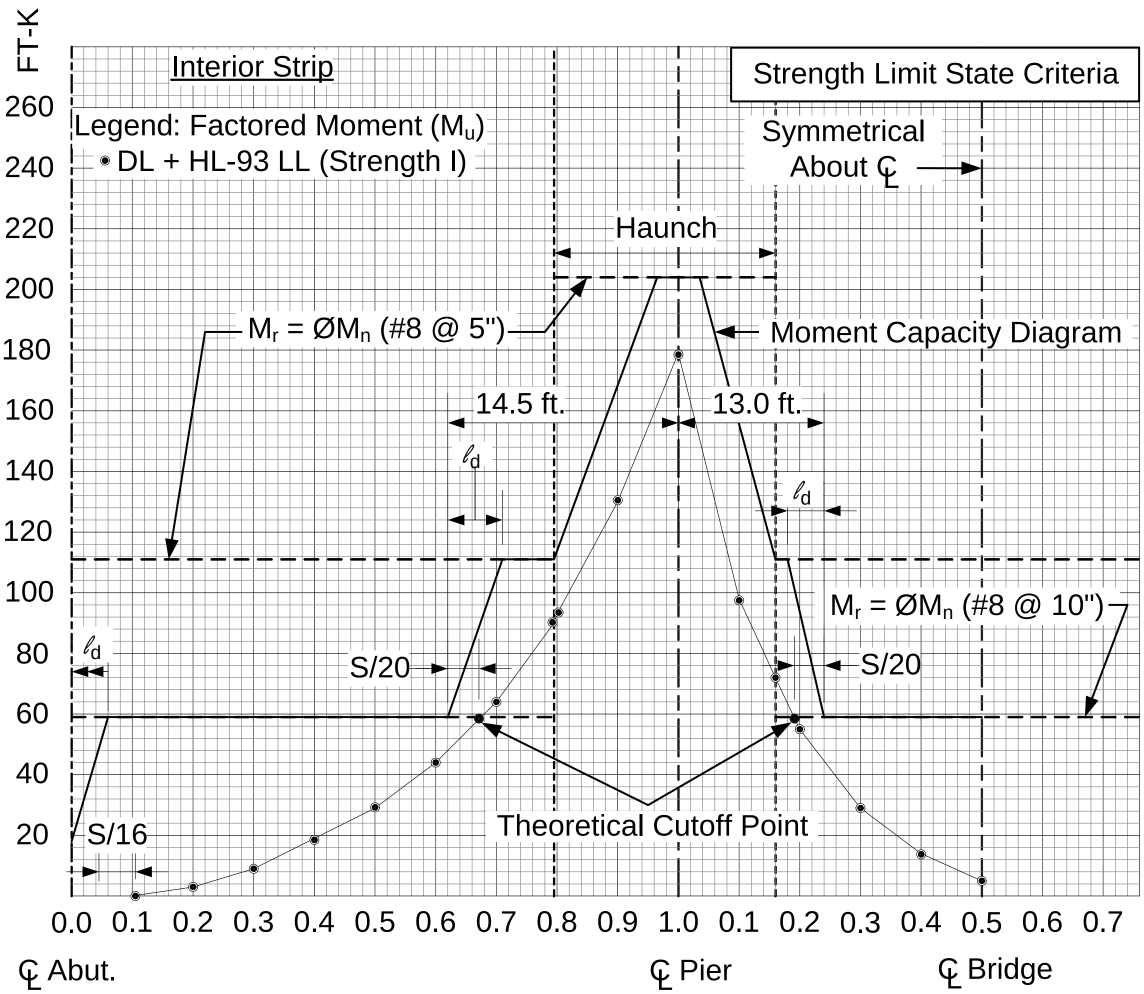

Figure E18.9

Negative Moment Cutoff Diagram

For same section depth and less steel, reinforcement will yield, therefore:

$\boxed{a = 1.38}$ in

$M_r = 104.9$ kip-ft (at C/L pier), $d_s := 25.5$ in

$M_r = 58.4$ kip-ft (in span), $d_s := 14.5$ in

The moment diagram equals the capacity of #8 at 10" at 12.5 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.11.1.2.1]**

$\frac{S}{20} = \frac{38}{20} = 1.9$ ft <u>controls</u> ℓ_d (#8) (See Table 9.9-2, Chapter 9)

Therefore, 1/2 of bars may be cut at 14.5 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.62 pt.)

E18-1.7.5.3.1 Fatigue Check (at Cutoff) - (0.62 Pt.)

Following the procedure in E18-1.7.5.2.1, using Fatigue I Limit State:

$f_s + f'_s \le 24 - 0.33 \cdot f_{min}$ (for $f_y = 60$ ksi)

Interpolating from Table E18.4, the moments at (0.62 pt.) of span 1 are:

$M_{DC} = 4.44$ kip-ft $M_{DW} = 0.4$ kip-ft

+Fatigue Truck = 13.7 kip-ft -Fatigue Truck = -8.68 kip-ft

For simplicity, assume fatigue criteria should be checked and use cracked section properties.

Calculate fatigue moment: $M_{fatigue} = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.5(\text{Fatigue Truck})$

$M_{fatigueMax} := 1.0(4.44) + 1.5(-8.68)$ $\boxed{M_{fatigueMax} = -8.58}$ kip-ft (tension)

$M_{fatigueMin} := 1.0(4.44) + 1.5(13.7)$ $\boxed{M_{fatigueMin} = 24.99}$ kip-ft (compression)

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile range.

Looking at values of $M_{fatigue}$, shows that the reinforcement goes through tensile and compressive stress during the fatigue cycle

See Figure E18.10, for definition of d_1, d_2, d', A_s and A'_s.

The moment arm used in equations below is: $(j_1)(d_1)$ for finding f_s
$(j_2)(d_2)$ for finding f'_s

Using: A_s = 0.94 in^2/ft, d_1 = 14.5 in, n = 8, and transformed section analysis, gives a value of j_1 = 0.915

Using: A'_s =1.71 in^2/ft, d_2 = 14.9 in, n = 8, and transformed section analysis, gives a value of j_2 = 0.893; k = x/d_2 = 0.33, where x = distance from compression face to neutral axis

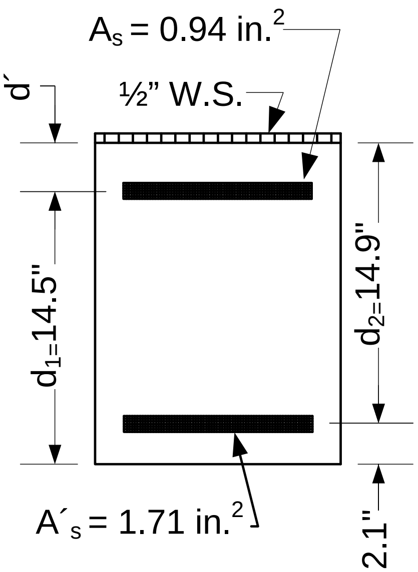

Figure E18.10

Cross Section - (0.62 pt.) Span 1

The tensile part of the stress range in the top bars is computed as:

$$f_s := \frac{M_{fatigueMax} \cdot 12}{A_s \cdot (j_1) \cdot d_1} \qquad \boxed{f_s = 8.26} \quad \text{ksi} \quad \text{(tension)}$$

The compressive part of the stress range in the top bars is computed as:

$$f'_s := \frac{M_{fatigueMin} \cdot 12}{A'_s \cdot (j_2) \cdot d_2} \cdot \frac{k - \left(\frac{d'}{d_2}\right)}{1 - k} \qquad \boxed{f'_s = -3.19} \quad \text{ksi} \quad \text{(compression)}$$

It is assumed (#9's at 7") is present at this location as compression steel (A'_s).

Therefore, total stress range on top steel:

$$f_s + f'_s = 8.26 - (-3.19) = 11.45 \quad \text{ksi}$$

$R_r := 24 - 0.33 \cdot f_{min}$ where $f_{min} = f'_s$, therefore: $\boxed{R_r = 25.05}$ ksi

Therefore, $f_s + f'_s$ = 11.45 ksi < R_r = 25.05 ksi <u>O.K..</u>

E18-1.7.5.3.2 Crack Control Check (at Cutoff) - (0.62 Pt.)

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\boxed{f_{r80\%} = 0.38}$ ksi $\boxed{c = 8.5}$ in $\boxed{I_g = 4913}$ in^4

$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$

Interpolating from Table E18.4, the moments at (0.62 pt.) of span 1 are:

$M_{DC} = 4.4$ kip-ft $M_{DW} = 0.4$ kip-ft $M_{LL+IM} = -5.88 + (-23.88) = -29.8$ kip-ft (LL#2)

$M_S := 1.0\cdot(4.4) + 1.0\cdot(-29.8)$ $\boxed{M_S = -25.4}$ kip-ft

M_{DW} (FWS) moment was ignored in order to obtain a greater tensile moment.

$$f_T = \frac{M_S \cdot c}{I_g} \qquad f_T := \frac{25.4\cdot(8.5)\cdot 12}{4913} \qquad \boxed{f_T = 0.53} \text{ ksi}$$

f_T =0.53 ksi > 80% f_r = 0.38 ksi; therefore, check crack control criteria

For: #8 at 10" c-c spacing ($A_S = 0.94\ in^2/ft$)

The values for γ_e , d_c , h , and β_S , used to calculate max. spacing (s) of reinforcement are :

$\gamma_e := 0.75$ for Class 2 exposure condition (top reinforcement)

$\boxed{d_c = 2.5}$ in $\boxed{h = 17}$ in $\boxed{\beta_S = 1.25}$

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.5.3.1, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 24.44}$ ksi < 0.6 f_y O.K. $$s \leq \frac{700\cdot(0.75)}{1.25\cdot(24.44)} - 2\cdot(2.50) = 17.2 - 5.0 = 12.2 \text{ in}$$

$s \leq 12.2$ in

Therefore, spacing prov'd. = 10 in < 12.2 in O.K.

E18-1.7.5.3.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 14.5 (ft) from the C/L of the pier. Remaining bars are extended beyond the point of inflection, a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater. **LRFD [5.11.1.2.3]**

$d_{eff} := 14.5$ in $\qquad$ ℓ_d (#8) (See Table 9.9-2, Chapter 9)

$$12 \cdot (d_b) = 12 \cdot (1.00) = 12.0 \text{ in}$$

$$\frac{S}{16} = \frac{38}{16} = 2.38 \text{ ft} \qquad \underline{\text{controls}}$$

Looking at the factored moment diagram (M_u) on Figure E18.9, the point of inflection is found at the (0.11 pt.). Therefore, the remaining bars could be terminated at 36.5 (ft) from the C/L of pier and these bars lapped with smaller size bars spaced at 10 inches.

Because this bar termination point is close to the abutment, run remaining bars (#8 at 10" c-c spacing) to the end of the slab.

E18-1.7.5.4 Span 2 Negative Moment Reinforcement (Cutoffs)

Capacities of #8 at 5" and #8 at 10" c-c spacing are stated in E18-1.7.5.3

The moment diagram equals the capacity of #8 at 10 " at 10.0 (ft) from the C/L of pier. Reinforcement shall be extended beyond this point a distance equal to the effective depth of the slab, 15 bar diameters, or 1/20 of the clear span, whichever is greater. **LRFD [5.11.1.2.1]**

$$\frac{S}{20} = \frac{51}{20} = 2.55 \text{ ft} \qquad \underline{\text{controls}}$$

ℓ_d (#8) (See Table 9.9-2, Chapter 9)

Therefore, 1/2 of bars may be cut at 13.0 (ft) from the C/L of pier if fatigue and crack control criteria are satisfied. (Check at 0.25 pt.)

E18-1.7.5.4.1 Fatigue Check (at Cutoff) - (0.25 Pt.)

Following the procedure in E18-1.7.5.2.1, using Fatigue I Limit State:

$$f_s + f'_s \le 24 - 0.33 \cdot f_{min} \qquad (\text{for } f_y = 60 \text{ ksi})$$

Interpolating from Table E18.4, the moments at (0.25 pt.) of span 2 are:

M_{DC} = -0.45 kip-ft $\qquad$ M_{DW} = -0.05 kip-ft

+Fatigue Truck = 10.9 kip-ft $\qquad$ -Fatigue Truck = -7.0 kip-ft

f_{ss} = tensile stress in steel reinforcement at the Service I Limit State (ksi) $\leq 0.6\ f_y$

The moment arm used to calculate f_{ss} is: (j) (h - d_c)
As shown in fatigue calculations in E18-1.7.5.3.1, j = 0.915

The value of f_{ss} and (s) are:

$\boxed{f_{ss} = 22.23}$ ksi $< 0.6\ f_y$ O.K. $\quad s \leq \frac{700 \cdot (0.75)}{1.25 \cdot (22.23)} - 2 \cdot (2.50) = 19.0 - 5.0 = 14.0$ in

$s \leq 14.0$ in

Therefore, spacing prov'd. = 10 in < 14.0 in O.K.

E18-1.7.5.4.3 Minimum Reinforcement Check

Following the procedure in E18-1.7.2.4, the minimum reinforcement check was found to be O.K.

Therefore, cut 1/2 of bars at 13.0 (ft) from the C/L of the pier. Remaining bars are extended beyond the point of inflection, a distance equal to the effective depth of the slab, 12 bar diameters, or 1/16 of the clear span, whichever is greater. **LRFD [5.11.1.2.3]**

$\frac{S}{16} = \frac{51}{16} = 3.19$ ft controls $\quad \ell_d$ (#8) (See Table 9.9-2, Chapter 9)

Looking at the factored moment diagram (M_u) on Figure E18.9, no point of inflection is found in span 2.

Therefore, run the remaining bars (#8 at 10" c-c spacing) to the C/L of span 2 and lap them.

E18-1.8 Evaluation of Longitudinal Reinforcement for Permit Vehicle

Check the adequacy of the longitudinal reinforcement to see if it has the moment capacity to handle factored moments due to applied dead load (including future wearing surface) and the Wisconsin Standard Permit Vehicle (Wis-SPV) (with a minimum gross vehicle load of 190 kips) on an interior strip. This requirement is stated in 17.1.2.1.

The Wisconsin Standard Permit Vehicle load that can be carried by the bridge is 225 kips, when the future wearing surface is present. Details for the calculation of this load are shown in Chapter 45, "Reinforced Concrete Slab Rating" example.

Wisconsin Standard Permit Vehicle (Wis-SPV) load capacity = 225 kips > 190 kips O.K.

E18-1.9 Longitudinal Reinforcement in Bottom of Haunch

At least (1/4) of maximum positive moment reinforcement in continuous-spans shall extend into the support **LRFD [5.11.1.2.2]**.

Max. positive (A_s) = $2.00 \frac{in^2}{ft}$ (#9 at 6" c-c spacing, in span 2)

Reinf. req'd. = $0.25 \cdot (2.00) = 0.5 \frac{in^2}{ft}$

Therefore, use #7 at 13 in. (0.55 in^2/ft) > reinf. req'd and min. reinf. on Standard 18.01 O.K.

See Figure E18.12 for a summary of longitudinal reinforcement selected and layout of transverse distribution steel selected in E18-1.12.

If $\frac{c}{d_s} \le 0.6$ for (f_y = 60 ksi) **LRFD [5.7.2.1]**, then reinforcement has yielded and the assumption is correct.

$\beta_1 := 0.85$ $\quad c := \frac{a}{\beta_1}$ $\quad \boxed{c = 1.93}$ in

$d_s := d_{neg}$ $\quad \boxed{d_s = 54.62}$ in

$\frac{c}{d_s} = 0.04 < 0.6$ therefore, the reinforcement will yield.

$$M_r = 0.90 \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

$$M_r := 0.9 \cdot (2.79) \cdot 60.0 \cdot \left(\frac{54.62 - \frac{1.64}{2}}{12}\right) \qquad \boxed{M_r = 675.5} \text{ kip-ft}$$

Therefore, $1.33(M_u)$ = 605.4 kip-ft < M_r = 675.5 kip-ft O.K.

E18-1.16.9 Crack Control Check for Transverse Slab Member

Check the negative moment reinforcement (at interior column).

This criteria shall be checked when tension (f_T) in the cross-section exceeds 80% of the modulus of rupture (f_r), specified in **LRFD [5.4.2.6]**.

Following the procedure in E18-1.7.1.3, using Service I Limit State:

$\boxed{f_r = 0.48}$ ksi $\quad \boxed{f_{r80\%} = 0.38}$ ksi $\quad \boxed{c = 29}$ in $\quad \boxed{I_g = 487780}$ in^4

$$M_s = 1.0(M_{DC}) + 1.0(M_{DW}) + 1.0(M_{LL+IM})$$

Using same moments selected for Strength Design in E18-1.16.6, at (interior column), provides:

M_{DC} = 15.2 kip-ft $\quad M_{DW}$ = 17.0 kip-ft $\quad M_{LL+IM}$ = 234.7 kip-ft

$$M_s := 1.0 \cdot (15.2) + 1.0(17.0) + 1.0 \cdot (234.7) \qquad \boxed{M_s = 266.9} \text{ kip-ft}$$

$$f_T = \frac{M_s \cdot c}{I_g} \qquad f_T := \frac{266.9 \cdot (29) \cdot 12}{487780} \qquad \boxed{f_T = 0.19} \text{ ksi}$$

f_T =0.19 ksi < 80% f_r = 0.38 ksi; therefore, crack control criteria check is not req'd.

Therefore, crack control criteria for transverse slab reinforcement is O.K.

Use: #5 at 9" c-c spacing for a 6.5 ft. transverse width over the pier.

The transverse slab member reinforcement (top/bottom), and the remainder of the transverse reinforcement is shown in Figure E18.21.

E18-1.17 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the remaining transverse reinforcement.

E18-1.17.1 Transverse Slab Member and Other Transverse Reinforcement

Following the procedure in E18-1.13.1:

All transverse slab member reinforcement (top/bottom) and remainder of transverse reinforcement in slab exceeds A_s req'd. for each face, and does not exceed maximum spacing.

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Welded wire fabric may be used for the vertical reinforcement. It must be deformed wire with a minimum size of D18.

Per **LRFD [5.8.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi} - 0.5V_s \right) \cot\theta$$

In the above equation, $\cot\theta$ is as defined in the V_c discussion above, and V_s is the shear reinforcement resistance at the section considered. Any lack of full reinforcement development shall be accounted for. Note that the reinforcement shown on the Standard Detail sheets satisfies these requirements.

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.7.3]**:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

LRFD [5.5.4.2] allows a ϕ factor equal to 0.9 for tension-controlled reinforced concrete sections such as the bridge deck.

The continuity reinforcement consists of mild steel reinforcement in the deck in the negative moment region over the pier. Consider both the non-composite and the superimposed dead loads and live loads for the Strength I design of the continuity reinforcement in the deck.

Moment resistance is developed in the same manner as shown in 19.3.3.13.1 for positive moments, except that the bottom girder flange is in compression and the deck is in tension. The moment resistance is formed by the couple resulting from the compression force in the bottom flange and the tension force from the longitudinal deck steel. Consider A_s to consist of the longitudinal deck steel present in the deck slab effective flange width as determined in 19.3.3.8. The distance, d_p, is taken from the bottom of the girder flange to the center of the longitudinal deck steel.

WisDOT exception to AASHTO:

Composite sections formed by WisDOT standard prestressed concrete girders shall be considered to be tension-controlled for the design of the continuity reinforcement. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required, and $\phi = 0.9$.

WisDOT policy item:

New bridge designs shall consider only the top mat of longitudinal deck steel when computing the continuity reinforcement capacity.

WisDOT policy item:

The continuity reinforcement shall be based on the greater of either the interior girder design or exterior girder and detailed as typical reinforcement for the entire width of the bridge deck. However, do not design the continuity steel based on the exterior girder design beneath a raised sidewalk. The continuity steel beneath a raised sidewalk should not be used for rating.

Based on the location of the neutral axis, the bottom flange compressive force may behave as either a rectangle or a T-section. On WisDOT standard prestressed girders, if the depth of the compression block, a, falls within the varying width of the bottom flange, the compression block acts as an idealized T-section. In this case, the width, b, shall be taken as the bottom flange width, and the width, b_w, shall be taken as the bottom flange width at the depth "a". During T-section behavior, the depth, h_f, shall be taken as the depth of the bottom flange of full width, b. See Figure 19.3-4 for details. Ensure that the deck steel is adequate to satisfy $M_r \geq M_u$.

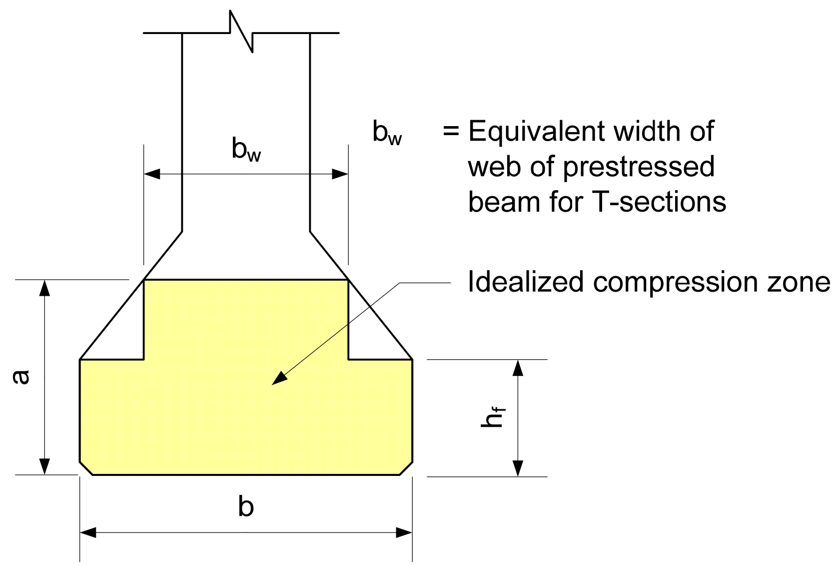

Figure 19.3-4
T-Section Compression Flange Behavior

The continuity reinforcement should also be checked to ensure that it meets the crack control provisions of **LRFD [5.7.3.4]**. This check shall be performed assuming severe exposure conditions. Only the superimposed loads shall be considered for the Service and Fatigue requirements.

The concrete between the abutting girder ends is usually of a much lesser strength than that of the girders. However, tests[1] have shown that, due to lateral confinement of the diaphragm concrete, the girder itself fails in ultimate negative compression rather than failure in the material between its ends. Therefore the ultimate compressive stress, f'_c, of the girder concrete is used in place of that of the diaphragm concrete.

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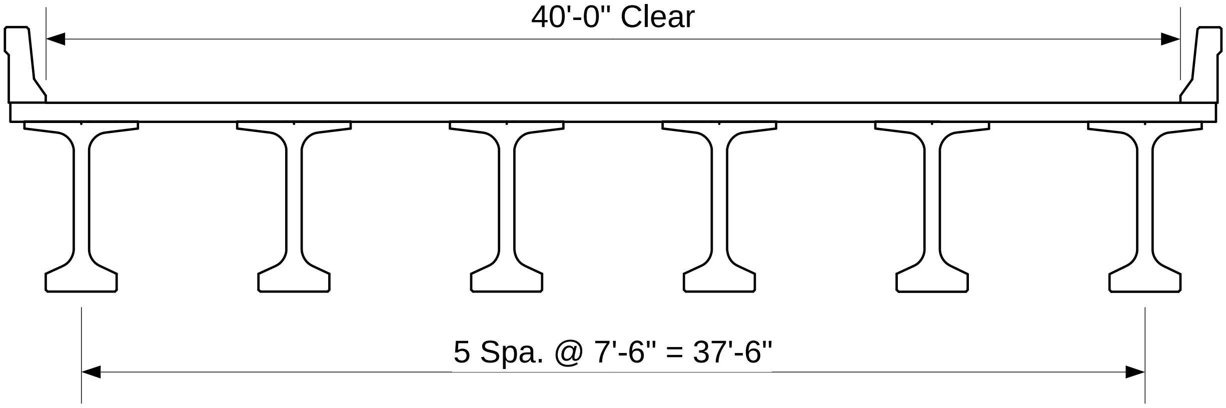
E19-1 Single Span Bridge, 72W" Prestressed Girders - LRFD

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 Sixth Edition - 2013 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

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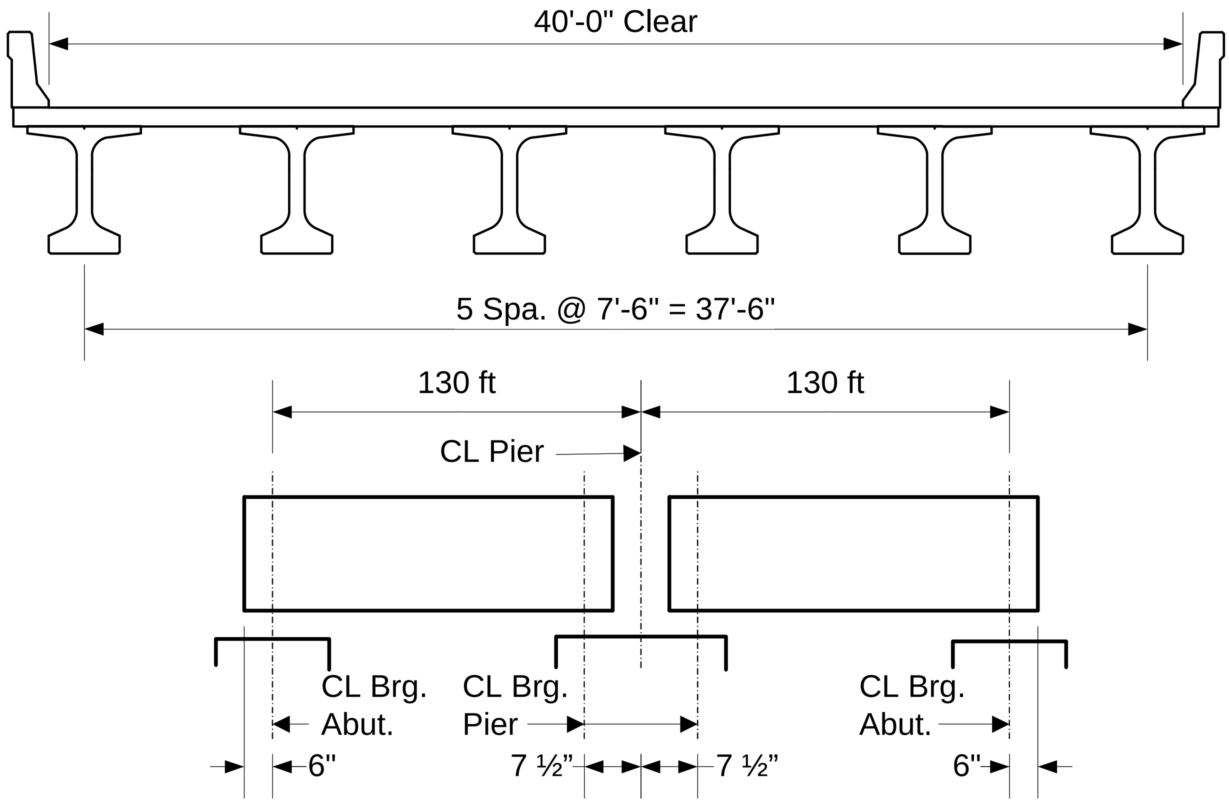
E19-2 Two-Span 54W" Girder, Continuity Reinforcement - LRFD

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 Sixth Edition - 2013 Interim)***

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

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 $f_s = f_y$

$$a := \frac{As \cdot f_y}{0.85 \cdot b_w \cdot f'_c} \qquad \boxed{a = 6.228} \text{ in}$$

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.

$$\beta_1 := 0.85 \qquad c := \frac{a}{\beta_1} \qquad \boxed{c = 7.327} \text{ in}$$

$\frac{c}{d_s} = 0.12 < 0.6$ 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]**:

$f_r := 0.37 \cdot \sqrt{f'_{cd}}$ $\boxed{f_r = 0.740}$ ksi

$M_{cr} = \gamma_3 \left(\gamma_1 \cdot f_r\right) S_c$

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

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 2635}$ kip-ft

$\boxed{1.33 \cdot M_u = 5796}$ kip-ft

Is M_r greater than the lesser value of M_{cr} and 1.33^*M_u? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho := \frac{As}{b_w \cdot d_e}$ $\boxed{\rho = 0.01170}$

$n := \frac{E_s}{E_B}$ $\boxed{n = 4.566}$

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n$ $\boxed{k = 0.278}$

$j := 1 - \frac{k}{3}$ $\boxed{j = 0.907}$

Note that the value of d_c should not include the 1/2-inch wearing surface.

$$d_c := cover - 0.5 + Bar_D(bar_{trans}) + \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_c = 3.19}$ in

$\boxed{M_{s1} = 2608}$ kip-ft

$$f_s := \frac{M_{s1}}{As \cdot j \cdot d_e} \cdot 12 \ \underline{<} \ 0.6\ f_y$$ $\boxed{f_s = 27.006}$ ksi $\underline{<}$ 0.6 f_y O.K.

The height of the composite section, h, is:

$$h := ht + hau + t_{se}$$ $\boxed{h = 63.500}$ in

$$\beta := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$ $\boxed{\beta = 1.076}$

$\gamma_e := 0.75$ for Class 2 exposure

$$S_{max} := \frac{700\gamma_e}{\beta \cdot f_s} - 2 \cdot d_c$$ $\boxed{S_{max} = 11.70}$ in

$\boxed{spa = 4.25}$ in

Is the bar spacing less than S_{max}? $\boxed{check = \text{"OK"}}$

Check the Fatigue 1 reinforcement limits in accordance with **LRFD [5.5.3]**:

$\gamma f_{LL} \cdot \Delta f \leq \Delta F_{TH}$ where $\Delta F_{TH} := 24 - 20\ \frac{f_{min}}{f_y}$

$\Delta F_{TH} := 24 - 0.33\ f_{min}$ (for f_y = 60 ksi)

f_{min} is equal to the stress in the reinforcement due to the moments from the permanent loads combined with the Fatigue I load combination. Δf is the stress range resulting from the fatigue vehicle.

Check stress in section for determination of use of cracked or uncracked section properties:

$$f_{top} := \frac{M_f}{S_c} \cdot 12$$ $\boxed{f_{top} = 0.328}$ ksi

$$f_{limit} := 0.095 \cdot \sqrt{f'_c}$$ $\boxed{f_{limit} = 0.269}$ ksi

Therefore: $\boxed{SectionProp = \text{"Cracked"}}$

If we assume the neutral axis is in the bottom flange, the distance from cracked section neutral axis to bottom of compression flange, y_{cr}, is calculated as follows:

$$\frac{b_w \cdot y_{cr}^2}{2} = n \cdot As \cdot (d_e - y_{cr})$$

$$y_{cr} := \frac{n \cdot As}{b_w} \cdot \left(\sqrt{1 + \frac{2 \cdot b_w \cdot d_e}{n \cdot As}} - 1 \right)$$ $\boxed{y_{cr} = 16.756}$ in <u>No Good</u>

Assume the neutral axis is in the web:

$t_{bf\_min} := 7.5$

$t_{bf\_max} := 15$ $\quad t_{taper} := t_{bf\_max} - t_{bf\_min}$ $\quad \boxed{t_{taper} = 7.500}$

$t_{web} := 7$ $\quad w_{taper} := b_w - t_w$ $\quad \boxed{w_{taper} = 23.500}$

$$(w_{taper}) \cdot t_{bf\_min} \cdot \left(x - \frac{t_{bf\_min}}{2} \right) + t_w \cdot \frac{x^2}{2} \ldots = 0$$
$$+ \left(\frac{w_{taper} \cdot t_{taper}}{2} \right) \cdot \left(x - t_{bf\_min} - \frac{t_{taper}}{3} \right) - n \cdot As \cdot (d_e - x)$$

CG of cracked section, $\boxed{x = 17.626}$ in

Cracked section moment of inertia:

$$I_{cr} := \frac{w_{taper} \cdot t_{bf\_min}^3}{12} + w_{taper} \cdot t_{bf\_min} \cdot \left(x - \frac{t_{bf\_min}}{2} \right)^2 + \frac{t_{web} \cdot x^3}{3} \ldots$$
$$+ \frac{w_{taper} \cdot t_{taper}^3}{36} + \frac{w_{taper} \cdot t_{taper}}{2} \cdot \left(x - t_{bf\_min} - \frac{t_{taper}}{2} \right)^2 + n \cdot As \cdot (d_e - x)^2$$

$\boxed{I_{cr} = 227583}$ in^4

Distance from centroid of tension reinforcement to the cracked section neutral axis:

$y_{rb} := d_e - x$ $\quad \boxed{y_{rb} = 42.685}$ in

$f_{min} := n \cdot \frac{M_f \cdot y_{rb}}{I_{cr}} \cdot 12$ $\quad \boxed{f_{min} = 10.913}$ ksi

$\Delta F_{TH} := 24 - 0.33 \cdot f_{min}$ (for f_y = 60 ksi) $\quad \boxed{\Delta F_{TH} = 20.399}$ ksi

$$\Delta f := n \cdot \frac{\left|M_{LLfatigue}\right| \cdot y_{rb}}{I_{cr}} \cdot 12$$ $\boxed{\Delta f = 3.488}$ ksi

$\boxed{\gamma f_{LL} \cdot \Delta f = 5.232}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{check = \text{"OK"}}$

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.

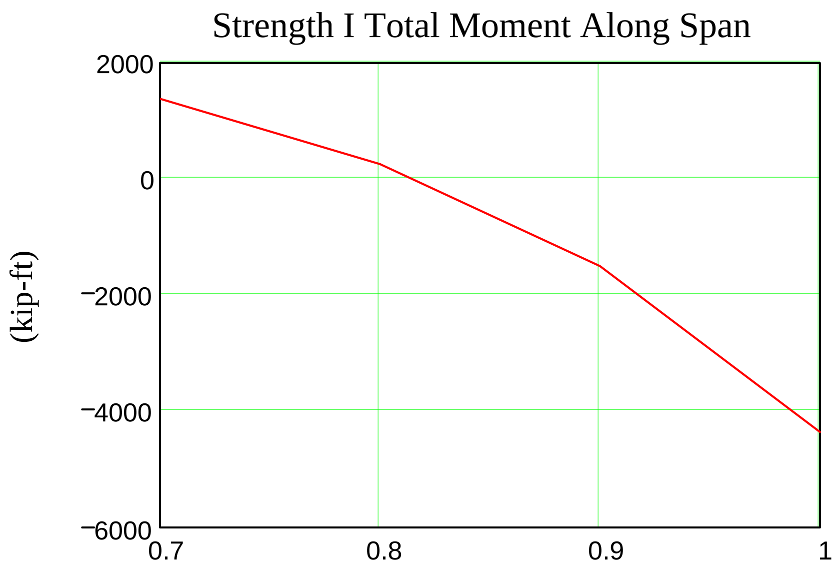
$$spa' := spa \cdot 2$$ $\boxed{spa' = 8.50}$ in

$$As' := \frac{As}{2}$$ $\boxed{As' = 10.588}$ in^2

$$a' := \frac{As' \cdot f_y}{0.85 \cdot b_w \cdot f'_c}$$ $\boxed{a' = 3.11}$ 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]**:

$\boxed{M_{cr} = 2635}$ kip-ft

$\boxed{1.33 \cdot Mu_{cut1} = 1996}$ kip-ft

Is $M_{r'}$ greater than the lesser value of M_{cr} and $1.33*Mu_{cut1}$? $\boxed{check = \text{"OK"}}$

Check the Service I crack control requirements in accordance with **LRFD [5.7.3.4]**:

$\rho' := \dfrac{As'}{b_w \cdot d_e}$ $\boxed{\rho' = 0.00585}$

$k' := \sqrt{(\rho' \cdot n)^2 + 2 \cdot \rho' \cdot n} - \rho' \cdot n$ $\boxed{k' = 0.206}$

$j' := 1 - \dfrac{k'}{3}$ $\boxed{j' = 0.931}$

$\boxed{Ms_{cut1} = 1565}$ kip-ft

$f_{s'} := \dfrac{Ms_{cut1}}{As' \cdot j' \cdot d_e} \cdot 12 \quad \le 0.6\ f_y$ $\boxed{f_{s'} = 31.582}$ ksi $\le 0.6\ f_y$ O.K.

$\boxed{\beta = 1.076}$

$\boxed{\gamma_e = 0.750}$

$S_{max'} := \dfrac{700\gamma_e}{\beta \cdot f_{s'}} - 2 \cdot d_c$ $\boxed{S_{max'} = 9.08}$ in

$\boxed{spa' = 8.50}$ in

Is the bar spacing less than $S_{max'}$? $\boxed{check = \text{"OK"}}$

Check the Fatigue 1 reinforcement limits in accordance with **LRFD [5.5.3]**:

The factored moments at the cut off are: $\boxed{Mf_{DLcut1} = 298}$ kip-ft

$\boxed{Mf_{LLcut1} = 458}$ kip-ft

$\boxed{Mfpos_{LLcut1} = 140}$ kip-ft

$Mf_{cut1} := Mf_{DLcut1} + Mf_{LLcut1}$ $\boxed{Mf_{cut1} = 756}$ kip-ft

Check stress in section for determination of use of cracked or uncracked section properties:

$f_{top\_cut1} := \dfrac{Mf_{cut1}}{S_c} \cdot 12$ $\boxed{f_{top\_cut1} = 0.234}$ ksi

$f_{limit} := 0.095 \cdot \sqrt{f'_c}$ $\boxed{f_{limit} = 0.269}$ ksi

Therefore: $\boxed{SectionProp = \text{"Un-Cracked"}}$

$f_{min\_cut1} := n \cdot \frac{Mf_{cut1}}{S_c} \cdot 12$ $\boxed{f_{min\_cut1} = 1.066}$ ksi

$\Delta F_{TH\_cut1} := 24 - 0.33 \cdot f_{min\_cut1}$ (for f_y = 60 ksi) $\boxed{\Delta F_{TH\_cut1} = 23.648}$ ksi

The live load range is the sum of the positive and negative fatigue moments:

$Mf_{LLrange} := Mf_{LLcut1} + Mfpos_{LLcut1}$ $\boxed{Mf_{LLrange} = 598}$ kip-ft

$\gamma fLL\Delta f\_cut1 := n \cdot \frac{Mf_{LLrange}}{S_c} \cdot 12$ $\boxed{\gamma fLL\Delta f\_cut1 = 0.844}$ ksi

Is $\gamma f_{LL} \cdot \Delta f$ less than ΔF_{TH}? $\boxed{check = \text{"OK"}}$

Therefore this cut off location, $\boxed{cut_1 = 0.90}$, is OK. The bar shall be extended past the cut off point a distance not less than the maximum of the following, **LRFD [5.11.1.2.3]**:

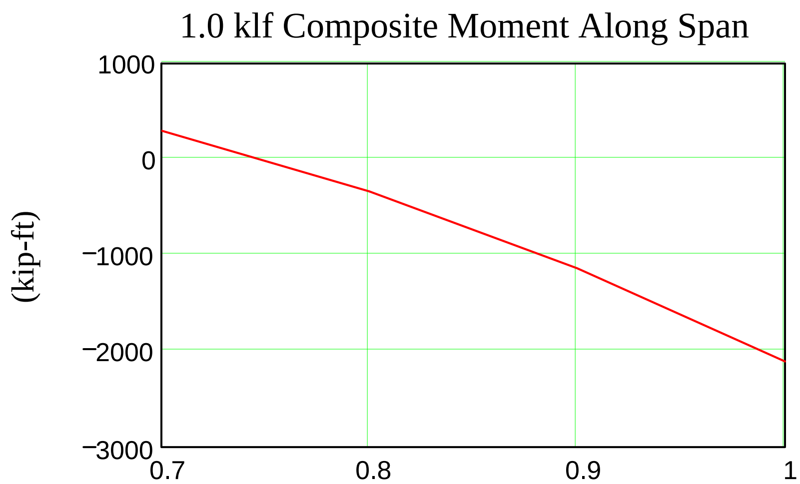
$$extend := \begin{pmatrix} d_e \\ 12 \cdot Bar_D(Bar_{No}) \\ 0.0625 \cdot L \cdot 12 \end{pmatrix} \qquad \boxed{extend = \begin{pmatrix} 60.311 \\ 13.536 \\ 97.500 \end{pmatrix}}$$

$$\boxed{\frac{max(extend)}{12} = 8.13} \text{ ft}$$

$X_1 := L \cdot (1 - cut_1) + \frac{max(extend)}{12}$ $\boxed{X_1 = 21.12}$ feet

USE $\boxed{X_1 = 22}$ feet from the CL of the pier.

The second bar cut off is located at the point of inflection under a uniform 1.0 klf composite dead load. At $cut_2 = 0.750$, $M_{cut2} = (79)$ kip-ft. Extend the bar the max(extend) distance calculated above past this point, or 4 feet past the first cut off, whichever is greater.



$X_{2a} := L\cdot(1 - cut_2) + \frac{max(extend)}{12}$ $\boxed{X_{2a} = 40.63}$ feet from the center of the pier

$X_{2b} := X_1 + 4$ $\boxed{X_{2b} = 26.00}$ feet from the center of the pier

$X_2 := max(X_{2a}, X_{2b})$ $\boxed{X_2 = 40.63}$ feet

USE $\boxed{X_2 = 41}$ feet from the CL of the pier.

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E19-3 Box Section Beam

This example shows design calculations for a single span prestressed box multi-beam bridge having a 2" concrete overlay and is designed for a 20 pound per square foot future wearing surface. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Sixth Edition - 2013 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:

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24.7 Composite Design

24.7.1 Composite Action

Composite action is present in steel girder superstructures when the steel beams or girders feature shear connectors which are embedded within the concrete slab. The shear connectors prevent slip and vertical separation between the bottom of the slab and the top of the steel member. Unless temporary shoring is used, the steel members deflect under the dead load of the wet concrete before the shear connectors become effective. However, since temporary shoring is not used in Wisconsin, composite action applies only to live loads and to portions of dead load placed after the concrete deck has hardened.

In the positive moment region, the concrete deck acts in compression and the composite section includes the slab concrete. However, in the negative moment region, the concrete deck acts in tension and the composite section includes the bar steel reinforcement in the slab.

As previously described, for LRFD, Wisconsin places shear connectors in both the positive and negative moment regions of continuous steel girder bridges, and both regions are considered composite in analysis and design computations. Negative flexure concrete deck reinforcement is considered in the section property calculations.

WisDOT policy item:

For rehabilitation projects, do not add shear studs in the negative moment region if none exist. Likewise, do not add additional studs in the positive moment region if shear connectors are provided and were designed for shear (not slab anchors on approximately 3’-0” to 4’-0” spacing).

If slab anchors are provided, consider as non-composite and add shear connectors if necessary for rating purposes. If adequate shear connector embedment into the deck is not achieved, additional reinforcement should be provided as per Figure 17.5-1.

24.7.2 Values of n for Composite Design

The effective composite concrete slab is converted to an equivalent steel area by dividing by n. For f'_c = 4 ksi, use n = 8.

f'_c = Minimum ultimate compressive strength of the concrete slab at 28 days

n = Ratio of modulus of elasticity of steel to that of concrete

The actual calculation of creep stresses in composite girders is theoretically complex and not necessary for the design of composite girders. Instead, a simple approach has been adopted for design in which a modular ratio appropriate to the duration of the load is used to compute the corresponding elastic section properties. As specified in **LRFD [6.10.1.1.1b]**, for transient loads applied to the composite section, the so-called ″short-term″ modular ratio, n, is used. However, for permanent loads applied to the composite section, the so-called ″long-term″ modular ratio, 3n, is used. The short-term modular ratio is based on the initial tangent modulus, E_c, of the concrete, while the long-term modular ratio is based on an effective apparent modulus, E_c/k, to account for the effects of creep. In U.S. practice, a value of k equal to 3 has been accepted as a reasonable value.

24.7.3 Composite Section Properties

The minimum effective slab thickness is equal to the nominal slab thickness minus 1/2" for wearing surface. The maximum effective slab width is defined in **LRFD [4.6.2.6]**.

24.7.4 Computation of Stresses

24.7.4.1 Non-composite Stresses

For non-composite sections, flexural stresses are computed using only non-composite (steel-only) section properties, as follows:

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

24.7.4.2 Composite Stresses

For composite sections, flexural stresses in the steel girder subjected to positive flexure are computed using appropriate non-composite (steel-only) and composite section properties, as follows:

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

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

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

Where:

f_b	=	Computed steel flexural stress
DLM	=	Dead lead moment
LLM	=	Live load moment
S	=	Elastic section modulus
DC1	=	DC dead load resisted by the steel section only (for example, steel girder, concrete deck, concrete haunch, cross-frames and stiffeners)

DC2 = DC dead load resisted by the composite section (for example, concrete parapets)

DW = Dead load due to future wearing surface and utilities

24.7.5 Shear Connectors

Refer to Standard for Plate Girder Details for shear connector details. Three shop or field welded 7/8" diameter studs at a length of 5" are placed on the top flange. The studs are equally spaced with a minimum clearance of 1 1/2" from the edge of the flange. On girders having thicker haunches where stud embedment is less than 2" into the slab, longer studs should be used to obtain the minimum embedment of 2".

Connectors which fall on the flange field splice plates should be repositioned near the ends of the splice plate. The maximum spacing of shear connectors is 2'. Connector spacings should begin a minimum of 9" from the centerline of abutments.

To determine the locations of shear connectors along the length of the girder, two general requirements must be satisfied:

- Spacing (or pitch) requirements governed by fatigue, as presented in **LRFD [6.10.10.1]**
- Number of connector requirements governed by strength, as presented in **LRFD [6.10.10.4]**

For the fatigue limit state, the pitch, p, of the shear connectors must satisfy the following equation:

$$p \leq \frac{nZ_r}{V_{sr}}$$

Where:

N = Number of shear connectors in a cross section

V_{sr} = Horizontal fatigue shear range per unit length (kips/in.)

Z_r = Shear fatigue resistance of an individual shear connector determined as specified in **LRFD [6.10.10.2]** (kips)

When computing the value for V_{sr}, the maximum value of composite moment of inertia in the span can be used.

For the strength limit state, the minimum number of required shear connectors, n, is computed for a given region according to the following equation:

$$n = \frac{P}{Q_r}$$

Where:

P = Total nominal shear force determined as specified in **LRFD [6.10.10.4.2]** (kips)

Q_r = Factored shear resistance of one shear connector (kips)

The given regions over which the required number of shear connectors is distributed are defined based on the point of maximum moment due to live load plus dynamic load allowance. This value is used because it applies to the composite section and is easier to locate than a maximum of the sum of all the moments acting on the composite section.

In most cases, the connector spacing (using three connectors per row) based on fatigue requirements is more than adequate for the strength design requirements. However for relatively long spans, additional shear connectors may be required to satisfy the strength design requirements.

In addition to the above general requirements, special shear connector requirements at points of permanent load contraflexure are presented in **LRFD [6.10.10.3]**.

Additional information and equations used for LRFD design of shear connectors are presented in **LRFD [6.10.10]**. In addition, a design example for shear connectors is also provided in this *Bridge Manual*.

24.7.6 Continuity Reinforcement

For continuous steel girder bridges, continuity reinforcement in the concrete deck must be considered in regions of negative flexure, as specified in **LRFD [6.10.1.7]**. Continuity reinforcement consisting of small bars with close spacing is intended to control concrete deck cracking.

If the longitudinal tensile stress in the concrete deck due to either the factored construction loads or the Service II load combination exceeds ϕf_r, then the following continuity reinforcement requirements must be satisfied:

- The total cross-sectional area of the longitudinal reinforcement in the deck shall be greater than or equal to one percent of the total cross-sectional area of the concrete deck.

- The required reinforcement shall be placed in two layers uniformly distributed across the deck width, with two-thirds being in the top layer and one-third in the bottom layer.

- The specified minimum yield strength, fy, of the reinforcing steel shall not be less than 60 ksi.

- The size of the reinforcement bars shall not exceed No. 6 bars.
- The spacing of the reinforcement bars shall not exceed 12 inches.

Tables 17.5-3 and 17.5-4 meet the criteria specified above.

In computing ϕf_r, f_r shall be taken as the modulus of rupture of the concrete (see **LRFD [5.4.2.6]**) and ϕ shall be taken as 0.90, which is the appropriate resistance factor for concrete in tension (see **LRFD [5.5.4.2.1]**). The longitudinal stresses in the concrete deck are computed as specified in **LRFD [6.10.1.1.1d]**. Superimposed dead loads and live loads are considered to be resisted by the composite section using the short-term modular ratio, n. Non-composite dead loads are supported by the girders alone.

Terminate the continuity reinforcement at the point of non-composite dead load contraflexure plus a development length. The bars are lapped to No. 4 bars.

For non-composite slabs in the negative moment region (on rehabilitation projects), extend the longitudinal reinforcement in Tables 17.5-3 and 17.5-4 a development length beyond the first shear connectors.

24.8 Field Splices

24.8.1 Location of Field Splices

Field splices shall be placed at the following locations whenever it is practical:

- At or near a point of dead load contraflexure for continuous spans
- Such that the maximum rolling length of the flange plates is not exceeded, thus eliminating one or more butt splices
- At a point where the fatigue in the net section of the base metal is minimized
- Such that section lengths between splices are limited to 120', unless special conditions govern

24.8.2 Splice Material

For homogeneous girders, the splice material is the same as the members being spliced. Generally, 3/4" diameter high strength A325 bolted friction-type connectors are used unless the proportions of the structure warrant larger diameter bolts.

24.8.3 Design

The following is a general description of the basic steps required for field splice design. These procedures and the accompanying equations are described in greater detail in **LRFD [6.13.6]**.

24.8.3.1 Obtain Design Criteria

The first design step is to identify the appropriate design criteria. This includes defining material properties, identifying relevant superstructure information and determining the splice location based on the criteria presented in 24.8.1.

Resistance factors used for field splice design are as presented in 17.2.6.

When calculating the nominal slip resistance of a bolt in a slip-critical connection, the value of the surface condition factor, K_s, shall be taken as follows:

- For structures that are to be painted, use Ks = 0.33.
- For unpainted, blast-cleaned steel or steel with organic zinc paint, use Ks = 0.50.

Where a section changes at a splice, the smaller of the two connected sections should be used in the design, as specified in **LRFD [6.13.6.1.1]**.

24.8.3.1.1 Section Properties Used to Compute Stresses

The section properties used to compute stresses are described in **LRFD [6.10.1.1.1]**.

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27.1 General

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports, ensuring that the bridge functions as intended. Bridges usually require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require greater consideration in minimizing forces caused by temperature change, friction and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

- Bridges are usually supported by reinforced concrete substructure units, and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as possible.

- Bridge bearings must be capable of withstanding and transferring dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

- Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance-free as possible.

WisDOT policy item:

WisDOT uses an installation temperature of 60°F for designing bearings. The temperature range considered for prestressed concrete girder superstructures is 5°F to 85°F, resulting in a range of 60° - 5° = 55° for bearing design. For prestressed girders an additional shrinkage factor of 0.0003 ft/ft should also be accounted for. The temperature range considered for steel girder superstructures is -30°F to 120°F, resulting in a range of 60° - (-30°) = 90° for bearing design.

WisDOT policy item:

According to **LRFD [14.4.1]**, the influence of dynamic load allowance need not be included for bearings. However, dynamic load allowance shall be included when designing bearings for bridges in Wisconsin. Apply dynamic load allowance in **LRFD [3.6.2]** to HL-93 live loads as stated in **LRFD [3.6.1.2, 3.6.1.3]** and distribute these loads, along with dead loads, to the bearings.

27.2 Bearing Types

Bridge bearings are of two general types: expansion and fixed. Bearings can be fixed in both the longitudinal and transverse directions, fixed in one direction and expansion in the other, or expansion in both directions. Expansion bearings provide for rotational movements of the girders, as well as longitudinal movement for the expansion and contraction of the bridge spans. If an expansion bearing develops a large resistance to longitudinal movement due to corrosion or other causes, this frictional force opposes the natural expansion or contraction of the span, creating a force within the span that could lead to a maintenance problem in the future. Fixed bearings act as hinges by permitting rotational movement, while at the same time preventing longitudinal movement. The function of the fixed bearing is to prevent the superstructure from moving longitudinally off of the substructure units. Both expansion and fixed bearings transfer lateral forces, as described in **LRFD [Section 3]**, from the superstructure to the substructure units. Both bearing types are set parallel to the direction of structural movement; bearings are not set parallel to flared girders.

When deciding which bearings will be fixed and which will be expansion on a bridge, several guidelines are commonly considered:

- The bearing layout for a bridge must be developed as a consistent system. Vertical movements are resisted by all bearings, longitudinal horizontal movements are resisted by fixed bearings and facilitated in expansion bearings, and rotations are generally allowed to occur as freely as possible.

- For maintenance purposes, it is generally desirable to minimize the number of deck joints on a bridge, which can in turn affect the bearing layout.

- The bearing layout must facilitate the anticipated thermal movements, primarily in the longitudinal direction, but also in the transverse direction for wide bridges.

- It is generally desirable for the superstructure to expand in the uphill direction, wherever possible.

- If more than one substructure unit is fixed within a single superstructure unit, then forces will be induced into the fixed substructure units and must be considered during design. If only one pier is fixed, unbalanced friction forces from expansion bearings will induce force into the fixed pier.

- For curved bridges, the bearing layout can induce additional stresses into the superstructure, which must be considered during design.

- Forces are distributed to the bearings based on the superstructure analysis.

A valuable tool for selecting bearing types is presented in **LRFD [Table 14.6.2-1]**, in which the suitability of various bearing types is presented in terms of movement, rotation and resistance to loads. In general, it is best to use a fixed or semi-expansion bearing utilizing an unreinforced elastomeric bearing pad whenever possible, provided adverse effects such as excessive force transfer to the substructure does not occur. Where a fixed bearing is required with greater rotational capacity, steel fixed bearings can be utilized. Laminated elastomeric bearings are the preferred choice for expansion bearings. When such expansion bearings fail to meet project requirements, steel Type "A-T" expansion bearings should be used. For curved and/or highly skewed bridges, consideration should be given to the use of pot bearings.

27.2.1 Elastomeric Bearings

Elastomeric bearings are commonly used on small to moderate sized bridges. Elastomeric bearings are either fabricated as plain bearing pads (consisting of elastomer only) or as laminated (steel reinforced) bearings (consisting of alternate layers of steel reinforcement and elastomer bonded together during vulcanization). A sample plain elastomeric bearing pad is illustrated in Figure 27.2-1, and a sample laminated (steel reinforced) elastomeric bearing is illustrated in Figure 27.2-2.

These bearings are designed to transmit loads and accommodate movements between a bridge and its supporting structure. Plain elastomeric bearing pads can be used for small bridges, in which the vertical loads, translations and rotations are relatively small. Laminated (steel reinforced) elastomeric bearing pads are often used for larger bridges with more sizable vertical loads, translations and rotations. Performance information indicates that elastomeric bearings are functional and reliable when designed within the structural limits of the material. See **LRFD [Section 14]**, *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition, Section 18, and *AASHTO M251* for design and construction requirements of elastomeric bearings.

WisDOT policy item:

WisDOT currently uses plain or laminated (steel reinforced) elastomeric bearings which are rectangular in shape. No other shapes or configurations are used for elastomeric bearings in Wisconsin.

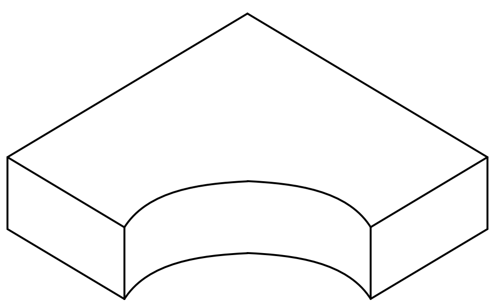

Figure 27.2-1
Plain Elastomeric Bearing

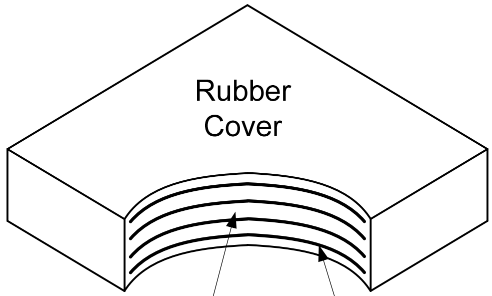

Figure 27.2-2
Laminated (Steel Reinforced) Elastomeric Bearing

AASHTO LRFD does not permit tapered elastomer layers in reinforced bearings. Laminated (steel reinforced) bearings must be placed on a level surface; otherwise gravity loads will produce shear strain in the bearing due to inclined forces. The angle between the alignment of the underside of the girder (due to the slope of the grade line, camber and dead load rotation) and a horizontal line must not exceed 0.01 radians, as per **LRFD [14.8.2]**. If the angle is greater than 0.01 radians or if the rotation multiplied by the top plate length is 1/8" or more, the 1 1/2" top steel plate must be tapered to provide a level load surface along the bottom of this plate under these conditions. The tapered plate will have a minimum thickness of 1 1/2" per *AASHTO Construction Specifications, Section 18*.

Plain and laminated (steel reinforced) elastomeric bearings can be designed by Method A as outlined in **LRFD [14.7.6]** and NCHRP-248 or by Method B as shown in **LRFD [14.7.5]** and NCHRP-298.

WisDOT policy item:

WisDOT uses Method A, as described in **LRFD [14.7.6]**, for elastomeric bearing design.

Method A results in a bearing with a lower capacity than a bearing designed using Method B. However, the increased capacity resulting from the use of Method B requires additional testing and quality control.

For several years, plain elastomeric bearing pads have performed well on prestressed concrete girder structures. Refer to the Standard for Bearing Pad Details for Prestressed Concrete Girders for details. Prestressed concrete girders using this detail are fixed into the concrete diaphragms at the supports, and the girders are set on 1/2" thick plain elastomeric bearing pads. Laminated (steel reinforced) bearing details and steel plate and elastomer thicknesses are given on the Standard for Elastomeric Bearings for Prestressed Concrete Girders.

The design of an elastomeric bearing generally involves the following steps:

1. Obtain required design input **LRFD [14.4 & 14.6]**

The required design input for the design of an elastomeric bearing at the service limit state is dead load, live load plus dynamic load allowance, minimum vertical force due to permanent load, and design translation. The required design input at the strength limit state is shear force. Other required design input is expansion length, girder or beam bottom flange width, minimum grade of elastomer, and temperature zone.

2. Select a feasible bearing type – plain or laminated (steel reinforced)

3. Select preliminary bearing properties **LRFD [14.7.6.2]**

   The preliminary bearing properties can be obtained from **LRFD [14.7.6.2]** or from past experience. The preliminary bearing properties include elastomer cover thickness, elastomer internal layer thickness, elastomer hardness, elastomer shear modulus, elastomer creep deflection, pad length, pad width, number of steel reinforcement layers, steel reinforcement thickness, steel reinforcement yield strength and steel reinforcement constant-amplitude fatigue threshold. WisDOT uses the following properties:

   - Elastomer cover thickness = 1/4"
   - Elastomer internal layer thickness = 1/2"
   - Elastomer hardness: Durometer 60 +/- 5
   - Elastomer shear modulus (G): 0.1125 ksi < G < 0.165 ksi
   - Elastomer creep deflection @ 25 years divided by instantaneous deflection = 0.30
   - Steel reinforcement thickness = 1/8"
   - Steel reinforcement yield strength = 36 ksi or 50 ksi
   - Steel reinforcement constant-amplitude fatigue threshold = 24 ksi

   However, not all of these properties are needed for a plain elastomeric bearing design.

4. Check shear deformation **LRFD [14.7.6.3.4]**

   Shear deformation, Δ_S, is the sum of deformation from thermal effects, Δ_{ST}, as well as creep and shrinkage effects, $\Delta_{Scr/sh}$ ($\Delta_S = \Delta_{ST} + \Delta_{Scr/sh}$).

   $$\Delta_{ST} = (\text{Expansion length})(\Delta_T)(\alpha)$$

   Where:

   Δ_T = Change in temperature (see 27.1) (degrees)

α	=	Coefficient of thermal expansion
	=	6×10^{-6} / °F for concrete, 6.5×10^{-6} / °F for steel

Shear deformation due to creep and shrinkage effects, $\Delta_{Scr/sh}$, should be added to Δ_{ST} for prestressed concrete girder structures. The value of $\Delta_{Scr/sh}$ is computed as follows:

$$\Delta_{Scr/sh} = (\text{Expansion length})(0.0003\ \text{ft/ft})$$

LRFD [14.7.6.3.4] provides shear deformation limits to help prevent rollover at the edges and delamination. The shear deformation, Δ_S, can be checked as specified in **LRFD [14.7.6.3.4]** and by the following equation:

$$h_{rt} \geq 2\,\Delta_S$$

Where:

h_{rt}	=	Smaller of total elastomer or bearing thickness (inches)
Δ_S	=	Maximum total shear deformation of the bearing at the service limit state (inches)

5. Check compressive stress **LRFD [14.7.6.3.2]**

The compressive stress, σ_S, at the service limit state can be checked as specified in **LRFD [14.7.6.3.2]** and by the following equations:

$\sigma_s \leq 0.80\ ksi$ and $\sigma_s \leq 1.00GS$ for plain elastomeric pads

$\sigma_s \leq 1.25\ ksi$ and $\sigma_s \leq 1.25GS$ for laminated (steel reinforced) elastomeric pads

Where:

σ_s	=	Service average compressive stress due to total load (ksi)
G	=	Shear modulus of elastomer (ksi)
S	=	Shape factor for the thickest layer of the bearing

LRFD [14.7.6.3.2] states that the stress limits may be increased by 10 percent where shear deformation is prevented, but this is not considered applicable to WisDOT bearings.

The shape factor for individual elastomer layers is the plan area divided by the area of the perimeter free to bulge. For laminated (steel reinforced) elastomeric bearings, the following requirements must be satisfied before calculating the shape factor:

- All internal layers of elastomer must be the same thickness.

- The thickness of the cover layers cannot exceed 70 percent of the thickness of the internal layers.

The shape factor, S_i, for rectangular bearings without holes can be determined as specified in **LRFD [14.7.5.1]** and by the following equation:

$$S_i = \frac{LW}{2h_{ri}(L + W)}$$

Where:

S_i = Shape factor for the i^{th} layer

h_{ri} = Thickness of i^{th} elastomeric layer in elastomeric bearing (inches)

L = Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)

W = Width of the bearing in the transverse direction (inches)

6. Check stability **LRFD [14.7.6.3.6]**

For stability, the total thickness of the rectangular pad must not exceed one-third of the pad length or one-third of the pad width as specified in **LRFD [14.7.6.3.6]**, or expressed mathematically:

$$H \le \frac{L}{3} \text{ and } H \le \frac{W}{3}$$

Where:

H = Total thickness of the elastomeric bearing (excluding top plate) (inches)

L = Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (inches)

W = Width of the bearing in the transverse direction (inches)

7. Check compressive deflection **LRFD [14.7.5.3.6, 14.7.6.3.3]**

The compressive deflection, δ, of the bearing shall be limited to ensure the serviceability of the deck joints, seals and other components of the bridge. Deflections of elastomeric bearings due to total load and to live load alone should be considered separately. Relative deflections across joints must be restricted so that a step doesn't occur at a deck joint. **LRFD [C14.7.5.3.6]** recommends that a maximum relative live load deflection across a joint be limited to 1/8".

WisDOT policy item:

WisDOT uses a live load + creep deflection limit of 1/8” for elastomeric bearing design.

Laminated (steel reinforced) elastomeric bearings have a nonlinear load deflection curve in compression. In the absence of information specific to the particular elastomer to be used, **LRFD [Figure C14.7.6.3.3-1]** may be used as a guide. Creep effects should be determined from information specific to the elastomeric compound used. Use the material properties given in this section. The compressive deflection, δ, can be determined as specified in **LRFD [14.7.5.3.6, 14.7.6.3.3]** and by the following equation:

$$\delta = \sum \varepsilon_i \, h_{ri}$$

Where:

δ	=	Instantaneous deflection (inches)
ε_i	=	Instantaneous compressive strain in the i^{th} elastomer layer of a laminated (steel reinforced) bearing
h_{ri}	=	Thickness of i^{th} elastomeric layer in a laminated (steel reinforced) bearing (inches)

Based on **LRFD [14.7.6.3.3]**, the initial compressive deflection of a plain elastomeric pad or in any layer of a laminated (steel reinforced) elastomeric bearing at the service limit state without dynamic load allowance shall not exceed $0.09h_{ri}$.

8. Check anchorage

WisDOT exception to AASHTO:

Design anchorage for laminated elastomeric bearings if the unfactored dead load stress is less than 200 psi. This is an exception to **LRFD [14.8.3]** based on past practice and good performance of existing bearings.

The factored force due to the deformation of an elastomeric element shall be taken as specified in **LRFD [14.6.3.1]** by the following equation:

$$H_u > GA \frac{\Delta_u}{h_{rt}}$$

Where:

H_u	=	Lateral load from applicable strength load combinations in **LRFD [Table 3.4.1-1]** (kips)

G	=	Shear modulus of the elastomer (ksi)
A	=	Plan area of elastomeric element or bearing (inches^2)
Δ_u	=	Factored shear deformation (inches)
h_{rt}	=	Total elastomer thickness (inches)

9. Check reinforcement **LRFD [14.7.5.3.5, 14.7.6.3.7]**

Reinforcing steel plates increase compressive and rotational stiffness, while maintaining flexibility in shear. The reinforcement must have adequate capacity to handle the tensile stresses produced in the plates as they counter the lateral bulging of the elastomer layers due to compression. These tensile stresses increase with compressive load. The reinforcement thickness must also satisfy the requirements of the *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition. The reinforcing steel plates can be checked as specified in **LRFD [Equation 14.7.5.3.5-1,2]**:

$$h_s \geq \frac{3\, h_{max}\, \sigma_s}{F_y} \text{ for service limit state}$$

$$h_s \geq \frac{2.0\, h_{max}\, \sigma_L}{\Delta F_{TH}} \text{ for fatigue limit state}$$

Where:

h_s	=	Thickness of the steel reinforcement (inches)
h_{max}	=	Thickness of the thickest elastomeric layer in elastomeric bearing (inches)
σ_s	=	Service average compressive stress due to total load (ksi)
Fy	=	Yield strength of steel reinforcement (ksi)
σ_L	=	Service average compressive stress due to live load (ksi)
ΔF_{TH}	=	Constant amplitude fatigue threshold for Category A as specified in **LRFD [6.6]** (ksi)

If holes exist in the reinforcement, the minimum thickness shall be increased by a factor equal to twice the gross width divided by the net width.

10. Rotation **LRFD [14.7.6.3.5]**

WisDOT exception to AASHTO:

Lateral rotation about the longitudinal axis of the bearing shall not be considered for straight girders.

WisDOT policy item:

Per **LRFD [14.8.2]**, a tapered plate shall be used if the inclination of the underside of the girder to the horizontal exceeds 0.01 radians. Additionally, if the rotation multiplied by the plate length is 1/8 inch or more, taper the plate.

27.2.2 Steel Bearings

For fixed bearings, a rocker plate attached to the girder is set on a masonry plate which transfers the girder reaction to the substructure unit. The masonry plate is attached to the substructure unit with anchor bolts. Pintles set into the masonry plate prevent the rocker from sliding off the masonry plate while allowing rotation to occur. This bearing is represented on the Standard for Fixed Bearing Details Type "A" - Steel Girders.

For expansion bearings, two additional plates are utilized, a stainless steel top plate and a Teflon plate allowing expansion and contraction to occur, but not in the transverse direction. This bearing is shown on the Standard for Stainless Steel - TFE Expansion Bearing Details Type "A-T".

Type "B" rocker bearings have been used for reactions greater than 400 kips and having a requirement for smaller longitudinal forces on the substructure unit. However, in the future, WisDOT plans to eliminate rocker bearings for new bridges and utilize pot bearings.

Pot and disc bearings are commonly used for moderate to large bridges. They are generally used for applications requiring a multi-directional rotational capacity and a medium to large range of load.

Hold down devices are additional details added to the Type "A-T" bearings for situations where live load can cause uplift at the abutment end of a girder. Ideally, proper span configurations would eliminate the need for hold down devices as they have proven to be a maintenance problem.

Since strength is not the governing criteria, anchor bolts are designed with Grade 36 steel for all steel bearings.

27.2.2.1 Type "A" Fixed Bearings

Type "A" Fixed Bearings prevent translation both transversely and longitudinally while allowing rotation in the longitudinal direction. This bearing is represented on the Standard for

Fixed Bearing Details Type "A" - Steel Girders. An advantage of this bearing type is that it is very low maintenance. See 27.2.2.2 Type "A-T" Expansion Bearings for design information.

27.2.2.2 Type "A-T" Expansion Bearings

Type "A-T" Expansion bearings are designed to translate by sliding an unfilled polytetrafluoroethylene (PTFE or TFE) surface across a smooth, hard mating surface of stainless steel. Expansion bearings of Teflon are not used without provision for rotation. A rocker plate is provided to facilitate rotation due to live load deflection or change of camber. The Teflon sliding surface is bonded to a rigid back-up material capable of resisting horizontal shear and bending stresses to which the sliding surfaces may be subjected.

Design requirements for TFE bearing surfaces are given in **LRFD [14.7.2]**. Stainless steel-TFE expansion bearing details are given on the Standard for Stainless Steel – TFE Expansion Bearing Details Type "A-T."

Friction values are given in the **LRFD [14.7.2.5]**; they vary with loading and temperature. It is permissible to use 0.10 for a maximum friction value and 0.06 for a minimum value when determining unbalanced friction forces.

The design of type "A-T" bearings is relatively simple. The first consideration is the rocker plate length which is proportional to the contact stress based on a radius of 24" using Grade 50W steel. The rocker plate thickness is determined from a minimum of 1 1/2" to a maximum computed from the moment by assuming one-half the bearing reaction value (N/2) acting at a lever arm of one-fourth the width of the Teflon coated plate (W/4) over the length of the rocker plate. The Teflon coated plate is designed with a minimum width of 7" and the allowable stress as specified in **LRFD [14.7.2.4]** on the gross area; in many cases this controls the capacity of the expansion bearings as given in the Standard for Stainless Steel – TFE Expansion Bearing Details Type "A-T."

The design of the masonry plate is based on a maximum allowable bearing stress as specified in **LRFD [14.8.1]**. The masonry plate thickness is determined from the maximum bending moments about the x-or y-axis using a uniform pressure distribution.

In lieu of designing specific bearings, the designer may use Service I limit state loading, including dynamic load allowance, and Standards for Fixed Bearing Details Type “A” – Steel Girders, Stainless Steel – TFE Expansion Bearing Details Type “A-T” and Steel Bearings for Prestressed Concrete Girders to select the appropriate bearing.

27.2.2.3 High-Load Multi-Rotational Bearings

High-Load Multi-Rotational bearings, such as pot or disc bearings, are commonly used for moderate to large bridges. They are generally used for curved and/or highly skewed bridge applications requiring a multi-directional rotational capacity and a medium to large range of load.

Pot bearings consist of a circular non-reinforced neoprene or rubber pad, of relatively thin section, which is totally enclosed by a steel pot. The rubber is prevented from bulging by the pot containing it and acts similar to a fluid under high pressure. The result is a bearing providing suitable rotation and at the same time giving the effect of a point-contact rocker bearing since the center of pressure does not vary more than 4 percent. As specified in **LRFD [14.7.4.1]**, the minimum vertical load on a pot bearing should not be less than 20 percent of the vertical design load.

Pot bearings resist vertical load primarily through compressive stress in the elastomeric pad. The pad can deform and it has some shear stiffness, but it has very limited compressibility. Pot bearings generally have a large reserve of strength against vertical load. Pot bearings facilitate rotation through deformation of the elastomeric pad. During rotation, one side of the pad compresses and the other side expands. Pot bearings can sustain many cycles of small rotations with little or no damage. However, they can experience significant damage when subjected to relatively few cycles of large rotations.

Pot bearings can also resist horizontal loads. They can either be fixed, guided or non-guided. Fixed pot bearings (see Figure 27.2-3) can not translate in any direction, and they resist horizontal load primarily through contact between the rim of the piston and the wall of the pot. Guided pot bearings (see Figure 27.2-4) can translate in only one direction, and they resist horizontal load in the other direction through the use of guide bars. Non-guided pot bearings (see Figure 27.2-5) can translate in any direction, and they do not resist horizontal loads in any direction.

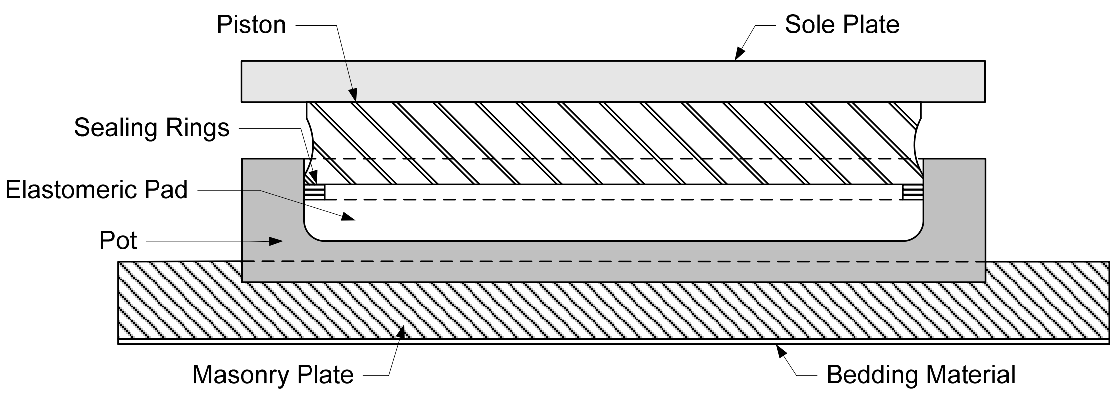

Figure 27.2-3
Fixed Pot Bearing

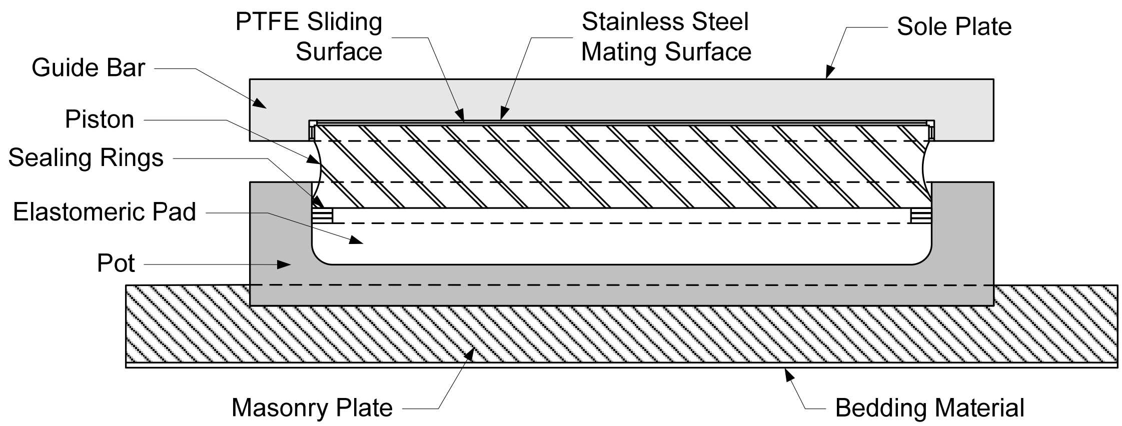

Figure 27.2-4
Guided Pot Bearing

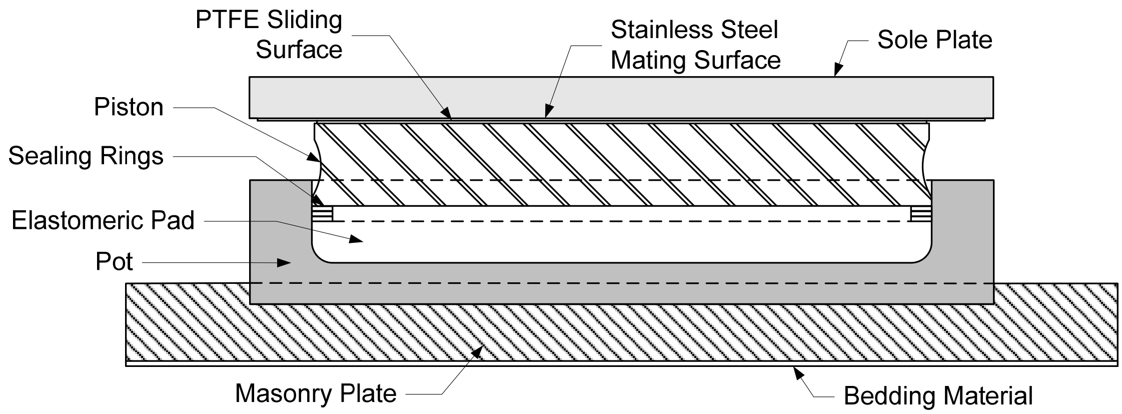

Figure 27.2-5
Non-Guided Pot Bearing

Disc bearings consist of a circular polyether urethane disc, confined by upper and lower steel plates and held in place by a positive location device. Limiting rings, either steel rings welded to the upper and lower steel plates or a circular recess in each of those plates, may also be used to partially confine the elastomer against lateral expansion. A shear-resisting mechanism shall be provided and it may be placed either inside or outside of the polyether urethane disc.

Disc bearings function by deformation of the polyether urethane disc, which should be stiff enough to resist vertical loads without excessive deformation and yet be flexible enough to accommodate the imposed rotations without liftoff or excessive stress on other components of the bearing assembly. The shear-resisting mechanism transmits horizontal forces between the upper and lower steel plates. As specified in **LRFD [14.7.8.4]**, the shear-resisting mechanism shall be capable of resisting a horizontal force in any direction equal to the larger

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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 Sixth Edition - 2013 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**.

E27-1.6 Stability

$H \le \frac{L}{3}$ and $H \le \frac{W}{3}$ **LRFD [14.7.6.3.6]**

$\boxed{H = 5.000}$ in

Bearing length check:

$L_{min} := 3 \cdot H$ $\boxed{L_{min} = 15}$ in

$\boxed{L = 10}$ in

Use the larger value: $\boxed{L = 15}$ in

Bearing width check:

$W_{min} := 3 \cdot H$ $\boxed{W_{min} = 15}$ in

$\boxed{W = 24}$ in

Use the larger value: $\boxed{W = 24}$ in

Revised shape factor and compressive stress for internal layer:

$\boxed{h_{ri} = 0.5}$ in

$\boxed{G = 0.1125}$ ksi

$S_i := \frac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ $\boxed{S_i = 9.231}$

$\boxed{1.25 \cdot G \cdot S_i = 1.298}$ ksi

$\sigma_s := \frac{DL_{serv} + LL_{serv}}{L \cdot W}$ $\boxed{\sigma_s = 0.636}$ ksi

$\boxed{\sigma_s = \text{"< 1.25GSi, OK"}}$

Check **LRFD [C14.7.6.1]**: $S_i^2 / n < 20$ (for rectangular shape with $n \geq 3$)

$S_i^2 / n = (9.231)^2 / 8 = 10.7 < 20$ "OK"

where n = (7 inter. layers + 1/2 (2 exter. layers)) = 8

Revised shape factor and compressive stress for the cover layer:

$\boxed{h_{rcover} = 0.25}$ in

$$S_{cover} := \frac{L \cdot W}{2 \cdot h_{rcover} \cdot (L + W)}$$

$\boxed{S_{cover} = 18.462}$ ksi

$\boxed{1.25 \cdot G \cdot S_{cover} = 2.596}$ ksi

$$\sigma_S := \frac{DL_{serv} + LL_{serv}}{L \cdot W}$$

$\boxed{\sigma_S = 0.636}$ ksi

$\boxed{\sigma_S = \text{"< 1.25GS, OK"}}$

E27-1.7 Compressive Deflection

LRFD [14.7.6.3.3, 14.7.5.3.6]

Average vertical compressive stress:

Average compressive stress due to total load

$\boxed{\sigma_S = 0.636}$ ksi

Average compressive stress due to live load

$$\sigma_L := \frac{LL_{serv}}{L \cdot W}$$

$\boxed{\sigma_L = 0.172}$ ksi

Average compressive stress due to dead load

$$\sigma_D := \frac{DL_{serv}}{L \cdot W}$$

$\boxed{\sigma_D = 0.464}$ ksi

Use **LRFD [Figure C14.7.6.3.3-1]** to estimate the compressive strain in the interior and cover layers. Average the values from the 50 Durometer and 60 Durometer curves to obtain values for 55 Durometer bearings.

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28.1 Introduction

Many structures have joints that must be properly designed and installed to insure their integrity and serviceability. Bridges as well as highway pavements, airstrips, buildings, etc. need joints to take care of expansion and contraction caused by temperature changes. However, bridges expand and contract more than pavement slabs or buildings and have their own special types of expansion devices.

Current practice is to limit the number of bridge expansion joints. This practice results in more movement at each joint. There are so many potential problems associated with joints that fewer joints are recommended practice. Expansion joints are placed on the high end of a bridge if only one joint is placed on the bridge. This is done to prevent the bridge from creeping downhill and to minimize the amount of water passing over the joint.

Open joints generally lead to future maintenance. Water and debris fall through the joint. Water running through an open joint erodes the soil under the structure, stains the bent cap and columns, and leads to corrosion of adjacent girders, diaphragms, and bearings. During freeze-thaw conditions, large icicles may form under the structure or ice may form on the roadway presenting a traffic hazard. Debris acts with water in staining the substructure units and plugs the drainage systems.

In the past, open steel finger type joints were used on long span bridges where large movements encountered. Finger joints were placed in the span near the point of contraflexure and were placed on the structure where they are required structurally. Drains were located to prevent drainage across the joint if feasible. In some areas, they were provided with a drainage trough to collect the water passing through.

Sliding steel plate joints are semi-open joints since water and light debris can pass through. A sealant placed in the joint prevents some water from passing through. It also prevents the accumulation of debris which can keep the joint from moving as it was designed. To date, considerable maintenance has occurred with sealants and neoprene troughs have been added to collect the water at some sites.

Currently finger and sliding plate details are maintained for joint maintenance and retrofitting but are not used for new structures. Watertight expansion devices such as strip seals and modular types are recommended for new structures. Although these expansion joints are not completely watertight; they have been effective in reducing damage to adjacent girders, diaphragms, bearings and substructure units.

The neoprene compression seal is a closed joint which is watertight if it is properly installed and an adequate adhesive is employed. Compression seals are only used for fixed joints. Strip-seals are watertight joints which are used in place of sliding plate joints or finger joints in an attempt to keep water and debris on the bridge deck surface.

Refer to Figure 12.7-1 for placement of expansion devices. The following criteria is used for placement of expansion devices:

28.2 Compression Seals

28.2.1 Description

This is a preformed, compartmented, elastomeric polychloropene (neoprene) device. In the past, compression seals were used sparingly on fixed joints provided there was little or no movement of the joint. However, compression seals shall no longer be used in this application due to the fact that the seals tend to leak over time. Compression seals shall be used only in longitudinal construction joint locations or for rehabilitation projects that do not involve full joint replacement (i.e., where the existing seal has pulled out of the joint and needs to be replaced).

28.2.2 Joint Design

Most applications have been for bridge rehabilitation where the seal is installed into the concrete deck without armor.

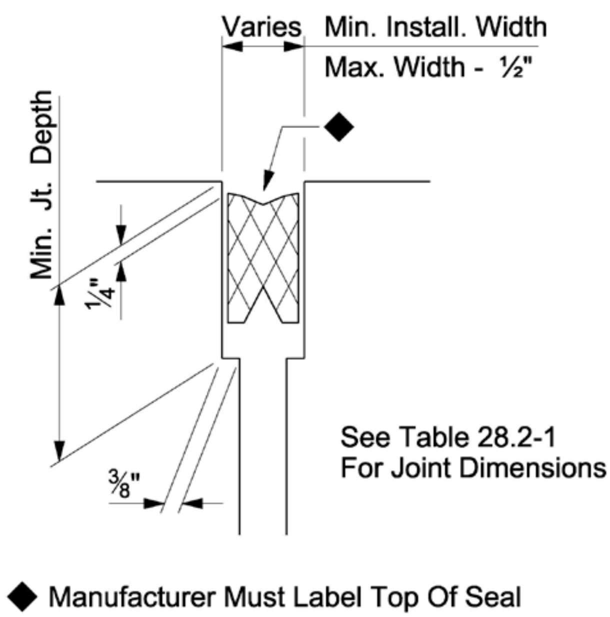

Figure 28.2-1 .
Joint Design

Manufacturer must label top of seal.

28.2.3 Seal Size

The width of the compression seal to be used in a given joint opening is computed by adding the as-constructed joint width plus a small width safety factor. For best results oversize the seal by a minimum of ½ inch. See Table 28.2-1 for approximate dimensions.

28.2.4 Installation

Ease of installation is achieved using a lubricant-adhesive which as the name implies acts initially as a lubricant then cures out to form an adhesive membrane between the contact faces of the angle and seal. This membrane, together with the compressive action of the seal, is designed to provide a waterproof joint interface.

The following information is a guide for the installation of neoprene compressive seals:

1. Thorough cleaning of joint faces is essential. Forced air or manual dusting handles most cases; use a solvent on oily areas.

2. Require application of the manufacturer's lubricant-adhesive to the sides of the neoprene seal as well as the joint faces. An adequate coating of the lubricant-adhesive is helpful in installation.

3. Proper installation tools consist of hand or machine tools that compress and eject the seal or weighted rollers that squeeze it in place. Screwdrivers, pry bars or other sharp ended tools which may puncture the seal are not allowed.

4. Stretching in excess of 5% is not permitted.

5. It is imperative that the seal be installed below the pavement surface. The minimum depth recess to top of seal is ¼ inch.

6. Prior to shipping, all compression seals are to be labeled TOP SIDE by the manufacturers. Field installation reports indicate difficulty in determining TOP SIDE for some types of seals. Also, the seal cross-section is not shown on a shop drawing unless the joint is armored.

28.2.5 Maintenance

Manual removal of incompressible materials which tend to collect within the joint opening is desirable. However, in most cases this is not necessary since the tire forces the material against the elastic neoprene seal which rebounds causing the material to bounce up and out of the seal.

It is essential to the operation of the seal that no form of hot or cold joint filler be placed over the top of the seal. This includes resurfacing mats or overlays. The reasons are as follows:

1. Hot fillers may either melt the seal or seriously affect the elastomeric properties for future performance.

2. The filler acts as a constant media of transmitting undue vertical tire forces to the compression seal which may break the interface bond.

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29.1 General

Wherever practical, bridge drainage should be carried off the structure along the curb or gutter line and collected with roadway catch basins. Floor drains are not recommended for structures less than 400' long and floor drain spacing is not to exceed 500' on any structure. However, additional floor drains are required on some structures due to flat grades, superelevations and the crest of vertical curves. The drains are spaced according to the criteria as set forth in 29.2. It should be understood that it is acceptable for water to be on the shoulder, or even half the traveled lane at lower speeds, during extreme rain events. Additional drains should not be provided other than what is required by design. Utilizing blockouts in parapets to facilitate drainage is not allowed.

Superelevation on structures often creates drainage problems other than at the low point especially if a reverse curve is involved. Water collects and flows down one gutter and as it starts into the superelevation transition it spreads out over the complete width of roadway at the point of zero cross-slope. From this point the water starts to flow into the opposite gutter. Certain freezing conditions can cause traffic accidents to occur in the flat area between the two transitions. To minimize the problem, locate the floor drain as close to the cross over point as practical. Floor drains are installed as near all joints as practical to prevent gutter flow from passing over and/or through the joints.

The Bureau of Structures recommends the Type “GC” floor drain for new structures. Type "GC" floor drains are gray iron castings that have been tested for hydraulic efficiency. Where hydraulic efficiency or girder flange to edge of deck geometry dictates the use of a different floor drain configuration, BOS recommends the Type “WF” floor drain. Steel fabricated floor drains Type “H” provide an additional 6” of downspout clearance and are retained for maintenance of structures where floor drain size modifications are necessary.

All of the floor drains shown on the Standards have grate inlets. When the longitudinal grade exceeds 1 percent, hydraulic flow testing indicates grates with rectangular longitudinal bars are more efficient than grates having transverse rectangular bars normal to flow. However, grates with bars parallel to the direction of traffic are hazardous to bicyclists and even motorcyclists as bar spacing is increased for hydraulic efficiency. As a result, transverse bars sloped toward the direction of flow are detailed for the cast iron floor drains.

Downspouts are to be fabricated from reinforced thermosetting resin (fiberglass) pipe having a diameter not less than 6” for all new structures. Galvanized standard pipe or reinforced fiberglass material may be used for downspouts when adjusting or rehabilitating existing floor drains. Downspouts are required on all floor drains to prevent water and/or chlorides from getting on the girders, bearings, substructure units, etc. Downspouts should be detailed to extend a minimum of 6” below low prestressed girder bottom flange or 1' below low steel to prevent flange or web corrosion. A downspout collector system is required on all structures over grade separations. Reinforced fiberglass pipe is recommended for all collector systems due to its durability and economy. In the design of collector systems, elimination of unnecessary bends and provision for an adequate number of clean outs is recommended.

29.2 Design Criteria

The flow of water in an open channel depends on its cross section, grade, and roughness. Generally, the gutter cross section on a structure is right triangular in shape with the curb, median or parapet forming the vertical leg. For design speeds 45 mph or less, floor drains are spaced at a distance such that the maximum gutter flow is restricted to a spread width of the shoulder plus one-half the adjacent through driving lane for a given design frequency storm. This defines the hypotenuse of the triangle if the shoulder and driving lane slope are equal. For design speeds greater than 45 mph, floor drains are spaced at a distance such that the maximum gutter flow is restricted to a spread width of the shoulder. An increase in longitudinal and transverse slope increases inlet capacity. In design, it is assumed that all of the water passing over the width of the inlet is taken by that inlet, the remaining water (Q bypass) continues to the next inlet.

For design a storm frequency of 10 years with a duration of 5 minutes is used. This gives an average rainfall intensity (i) of approximately 6" per hour in Wisconsin. A run-off coefficient (C) of 0.9 is used for concrete surfaces.

The Rational Method (English Units) converts rainfall intensity for a given design frequency storm to run-off by the following equation:

$$Q = C\,i\,A$$

Where:

Q	=	peak rate of run-off in cfs.
C	=	run-off coefficient for surface type.
i	=	rainfall intensity in inches/hour.
A	=	drainage area in acres = $\frac{LW}{43560}$

Where:

L	=	floor drain spacing in feet.
W	=	contributing structure width in feet.

The Manning equation modified for triangular flow is used to compute Q and Q_{bypass} for the given gutter section. The modified equation is:

$$Q = 0.56\left(\frac{Z}{n}\right)(S_o)^{\frac{1}{2}}(d)^{\frac{8}{3}}$$

Where:

Q	=	discharge in cfs.
Z	=	reciprocal of cross slope.
n	=	Manning's coefficient of roughness, use n = 0.014 for concrete.
S_o	=	longitudinal slope in feet/foot.
d	=	depth of flow at the deepest point (gutter line) in feet.

Refer to Table 29.2-1, Table 29.2-2 and Table 29.2-3 for values of (Z/n) and to Figure 29.2-1 for a nomographic solution to the Manning equation.

CROSS SLOPE, Sc			1/Sc	VALUES OF Z/n				
				n				
in/ft	in/ft	ft/ft	Z	0.012	0.013	0.014	0.015	0.016
	0.0120	0.0010	1000.00	83,333	76,923	71,429	66,667	62,500
1/64	0.0156	0.0013	768.00	64,000	59,077	54,857	51,200	48,000
	0.0240	0.0020	500.00	41,667	38,462	35,714	33,333	31,250
1/32	0.0313	0.0026	384.00	32,000	29,538	27,429	25,600	24,000
	0.0360	0.0030	333.33	27,778	25,641	23,810	22,222	20,833
	0.0480	0.0040	250.00	20,833	19,231	17,857	16,667	15,625
	0.0600	0.0050	200.00	16,667	15,385	14,286	13,333	12,500
1/16	0.0625	0.0052	192.00	16,000	14,769	13,714	12,800	12,000
	0.0720	0.0060	166.67	13,889	12,821	11,905	11,111	10,417
	0.0840	0.0070	142.86	11,905	10,989	10,204	9,524	8,929
3/32	0.0938	0.0078	128.00	10,667	9,846	9,143	8,533	8,000
	0.0960	0.0080	125.00	10,417	9,615	8,929	8,333	7,813
	0.1000	0.0083	120.00	10,000	9,231	8,571	8,000	7,500
	0.1080	0.0090	111.11	9,259	8,547	7,937	7,407	6,944
	0.1200	0.0100	100.00	8,333	7,692	7,143	6,667	6,250
1/8	0.1250	0.0104	96.00	8,000	7,385	6,857	6,400	6,000
	0.1320	0.0110	90.91	7,576	6,993	6,494	6,061	5,682
	0.1440	0.0120	83.33	6,944	6,410	5,952	5,556	5,208
5/32	0.1563	0.0130	76.80	6,400	5,908	5,486	5,120	4,800
	0.1680	0.0140	71.43	5,952	5,495	5,102	4,762	4,464
	0.1800	0.0150	66.67	5,556	5,128	4,762	4,444	4,167
3/16	0.1875	0.0156	64.00	5,333	4,923	4,571	4,267	4,000
	0.1920	0.0160	62.50	5,208	4,808	4,464	4,167	3,906
	0.2000	0.0167	60.00	5,000	4,615	4,286	4,000	3,750
	0.2040	0.0170	58.82	4,902	4,525	4,202	3,922	3,676
	0.2160	0.0180	55.56	4,630	4,274	3,968	3,704	3,472
7/32	0.2188	0.0182	54.86	4,571	4,220	3,918	3,657	3,429
	0.2280	0.0190	52.63	4,386	4,049	3,759	3,509	3,289
	0.2400	0.0200	50.00	4,167	3,846	3,571	3,333	3,125
1/4	0.2500	0.0208	48.00	4,000	3,692	3,429	3,200	3,000
9/32	0.2813	0.0234	42.67	3,556	3,282	3,048	2,844	2,667
19/64	0.2969	0.0247	40.42	3,368	3,109	2,887	2,695	2,526
	0.3000	0.0250	40.00	3,333	3,077	2,857	2,667	2,500

Table 29.2-1
Values of Z/n for Manning's Equation

CROSS SLOPE, Sc			1/Sc	VALUES OF Z/n				
				n				
in/ft	in/ft	ft/ft	Z	0.012	0.013	0.014	0.015	0.016
5/16	0.3125	0.0260	38.40	3,200	2,954	2,743	2,560	2,400
21/64	0.3281	0.0273	36.57	3,048	2,813	2,612	2,438	2,286
11/32	0.3438	0.0286	34.91	2,909	2,685	2,494	2,327	2,182
	0.3600	0.0300	33.33	2,778	2,564	2,381	2,222	2,083
3/8	0.3750	0.0313	32.00	2,667	2,462	2,286	2,133	2,000
	0.4000	0.0333	30.00	2,500	2,308	2,143	2,000	1,875
13/32	0.4063	0.0339	29.54	2,462	2,272	2,110	1,969	1,846
	0.4200	0.0350	28.57	2,381	2,198	2,041	1,905	1,786
7/16	0.4375	0.0365	27.43	2,286	2,110	1,959	1,829	1,714
15/32	0.4688	0.0391	25.60	2,133	1,969	1,829	1,707	1,600
	0.4800	0.0400	25.00	2,083	1,923	1,786	1,667	1,563
1/2	0.5000	0.0417	24.00	2,000	1,846	1,714	1,600	1,500
17/32	0.5313	0.0443	22.59	1,882	1,738	1,613	1,506	1,412
	0.5400	0.0450	22.22	1,852	1,709	1,587	1,481	1,389
9/16	0.5625	0.0469	21.33	1,778	1,641	1,524	1,422	1,333
19/32	0.5938	0.0495	20.21	1,684	1,555	1,444	1,347	1,263
	0.6000	0.0500	20.00	1,667	1,538	1,429	1,333	1,250
5/8	0.6250	0.0521	19.20	1,600	1,477	1,371	1,280	1,200
21/32	0.6563	0.0547	18.29	1,524	1,407	1,306	1,219	1,143
	0.6600	0.0550	18.18	1,515	1,399	1,299	1,212	1,136
11/16	0.6875	0.0573	17.45	1,455	1,343	1,247	1,164	1,091
	0.7000	0.0583	17.14	1,429	1,319	1,224	1,143	1,071
23/32	0.7188	0.0599	16.69	1,391	1,284	1,192	1,113	1,043
	0.7200	0.0600	16.67	1,389	1,282	1,190	1,111	1,042
3/4	0.7500	0.0625	16.00	1,333	1,231	1,143	1,067	1,000
	0.7800	0.0650	15.38	1,282	1,183	1,099	1,026	962
25/32	0.7812	0.0651	15.36	1,280	1,182	1,097	1,024	960
	0.8000	0.0667	15.00	1,250	1,154	1,071	1,000	938
13/16	0.8125	0.0677	14.77	1,231	1,136	1,055	985	923
	0.8400	0.0700	14.29	1,190	1,099	1,020	952	893
27/32	0.8438	0.0703	14.22	1,185	1,094	1,016	948	889
	0.8500	0.0708	14.12	1,176	1,086	1,008	941	882
7/8	0.8750	0.0729	13.71	1,143	1,055	980	914	857

Table 29.2-2

Values of Z/n for Manning’s Equation

CROSS SLOPE, Sc			1/Sc	VALUES OF Z/n				
				n				
in/ft	in/ft	ft/ft	Z	0.012	0.013	0.014	0.015	0.016
	0.9000	0.0750	13.33	1,111	1,026	952	889	833
39/32	1.2188	0.1016	9.85	821	757	703	656	615
15/16	0.9375	0.0781	12.80	1,067	985	914	853	800
	0.9500	0.0792	12.63	1,053	972	902	842	789
	0.9600	0.0800	12.50	1,042	962	893	833	781
31/32	0.9688	0.0807	12.39	1,032	953	885	826	774
1	1.000	0.0833	12.00	1,000	923	857	800	750
	1.020	0.0850	11.76	980	905	840	784	735
	1.080	0.0900	11.11	926	855	794	741	694
	1.140	0.0950	10.53	877	810	752	702	658
	1.200	0.1000	10.00	833	769	714	667	625
2	2.000	0.1667	6.000	500	462	429	400	375
	2.400	0.2000	5.000	417	385	357	333	313
3	3.000	0.2500	4.000	333	308	286	267	250
	3.600	0.3000	3.333	278	256	238	222	208
4	4.000	0.3333	3.000	250	231	214	200	188
	4.800	0.4000	2.500	208	192	179	167	156
5	5.000	0.4167	2.400	200	185	171	160	150
6	6.000	0.5000	2.000	167	154	143	133	125
7	7.000	0.5833	1.714	143	132	122	114	107
	7.200	0.6000	1.667	139	128	119	111	104
8	8.000	0.6667	1.500	125	115	107	100	94
	8.400	0.7000	1.429	119	110	102	95	89
9	9.000	0.7500	1.333	111	103	95	89	83
	9.600	0.8000	1.250	104	96	89	83	78
10	10.00	0.8333	1.200	100	92	86	80	75
	10.80	0.9000	1.111	93	85	79	74	69
11	11.00	0.9167	1.091	91	84	78	73	68
	11.50	0.9583	1.043	87	80	75	70	65
12	12.00	1.0000	1.000	83	77	71	67	63

Table 29.2-3
Values of Z/n for Manning's Equation

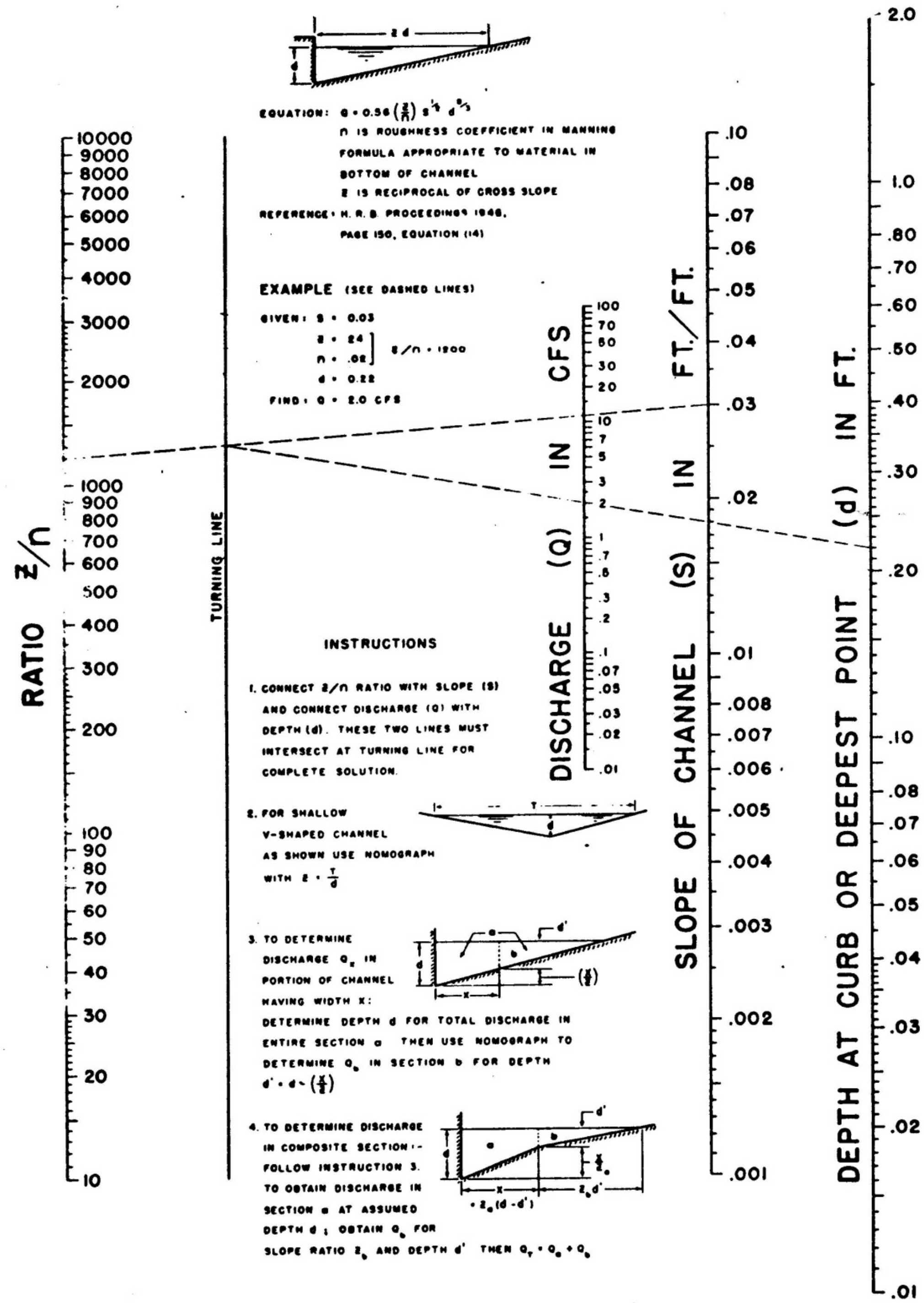

Figure 29.2-1
Nomograph for Flow in Triangular Channels
Modified Manning Solution

29.3 Design Example

The following method is used to compute floor drain spacing by equating net discharge to the Rational Method:

Given: Structure 1200 feet long on a 0.3% grade having a cross slope of 0.02 feet/foot with a contributing structure width of 23'-6". Use Type "GC" floor drain.

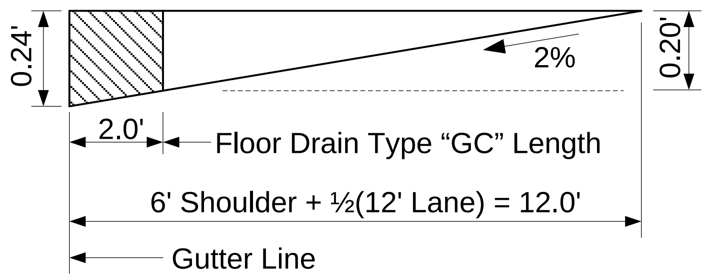

Figure 29.3-1
Cross Section of Flow

Compute: Floor drain spacing

From Table 29.2-1 with a cross slope of 0.02 feet/foot

$(Z/n) = 3571.$

From Figure 29.2-1, $Q = 2.44$ cfs and $Q_{bypass} = 1.50$ cfs.

$$L = \left(Q - Q_{bypass}\right)\frac{43560}{CiW}$$

$$L = \left(2.44 - 1.5\right)\frac{43560}{0.9 \cdot 6 \cdot 23.5}$$

$L = 323$ ft

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30.1 Crash-Tested Bridge Railings and FHWA Policy

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, 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, ornamental protective screening, vertical face concrete parapets with combination type C1-C6 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.

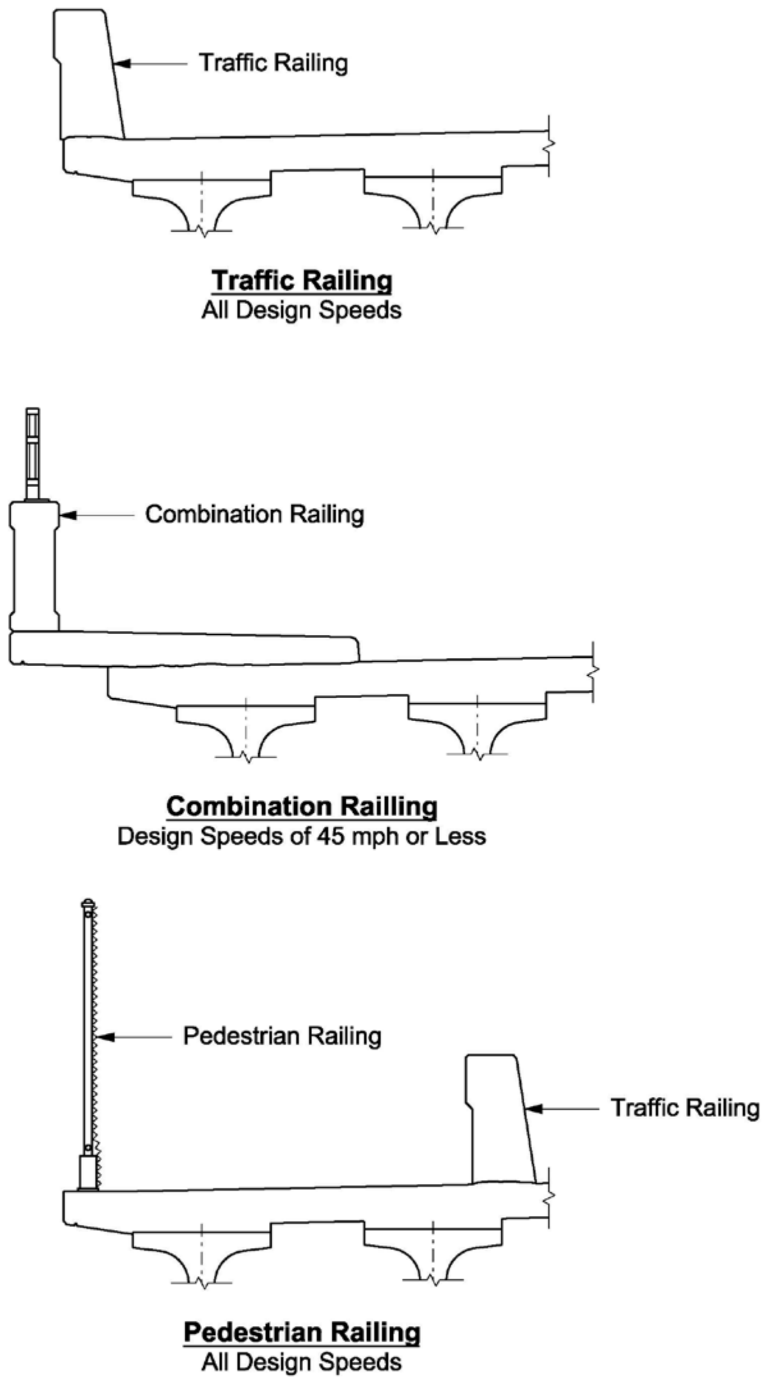


Figure 30.2-1
Bridge Railing Types

The application of bridge railings shall comply with the following guidance:

1. All bridge railings shall conform to **LRFD [13]**.

2. For all new bridge plans with a PS&E date after 2013, Traffic Railings placed on structures with a design speed exceeding 45 mph shall be solid concrete parapets. Where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints, designer shall contact the Bureau of Structures Development Section to receive approval for an exception to this policy.

3. Traffic Railings placed on structures with a design speed of 45 mph or less can be either solid concrete parapets or open railings with the exception as noted below in the single slope parapet application section.

4. New bridge plans utilizing concrete parapets shall be designed with "SS" ("32SS", "36SS", "42SS", or "56SS") parapets.

5. Per **LRFD [13.7.3.2]**, the minimum Traffic Railing height shall be 27" for TL-3, 32" for TL-4, 42" for TL-5, and 90" for TL-6. The railing applications as noted below meet these requirements.

6. Per **LRFD [13.8.1]** and **LRFD [13.9.2]**, the minimum height of a Pedestrian (and/or bicycle) Railing shall be 42" measured from the top of the walkway or riding surface respectively. Per the *Wisconsin Bicycle Facility Design Handbook*, on bridges that are signed or marked as bikeways and bicyclists are operating right next to the railing, the preferred height of the railing is 54". The higher railing/parapet height is especially important and should be used on long bridges, high bridges, and bridges having high bicyclist volumes. If an open railing is used, the clear opening between horizontal elements shall be 6 inches or less.

7. Aesthetics associated with bridge railings shall follow guidance provided in Section 30.4.

The designation for railing types are shown on the Standard Details. Bridge railings shall be employed as follows:

1. The single slope parapet "32SS" shall be used as a Traffic Railing on all structures with a design speed exceeding 45 mph. The "36SS" and "42SS" parapets should be used where the Region determines that there is high truck traffic and/or curved horizontal alignment creating more potential for overtopping the parapet, or if roadway concrete barrier single slope (CBSS) of the same height adjoins the bridge. Single slope parapet "56SS" shall only used if 56" CBSS adjoins the bridge. The "SS" parapets were crash-tested per NCHRP Report 350 specifications and meet crash test criteria for TL-4.

   A "SS" or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and "HF" parapets shall be used as Traffic Railings for rehabilitation projects only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The "51F" parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. The vertical face parapet "A" can be used for all design speeds. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. Under some circumstances, the vertical face parapet "A" can be used as a Traffic Railing for design speeds exceeding 45 mph with the approval of the Bureau of Structures Development Section. The vertical face parapet "A" was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic railings may be used if crash tested according to Section 30.1 or follow the guidance provided in Section 30.4.

   The Texas style aesthetic parapet, type "TX", can be used as a Combination Railing or Traffic 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 has been used sparingly and shall not be used on new structures with a PS&E date 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. This railing is similar to the type "F" railing with two main differences: the height of this rail meets the requirements for pedestrian facilities and it is a solid rail type that could be used on a grade separation structure. The type "PF" 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.

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. 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 railing meets crash test criteria for TL-4).

8. Chain Link Fence and Ornamental Protective 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, Ornamental Protective 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. 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.

14. 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*.

15. 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.

<u>*Note: WisDOT is currently investigating alternative open railing types for future use on bridge structures in Wisconsin. Specifically, new rail standards to replace the existing type “W” and type “M” railings are being considered.*</u>

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

10'-0" clear between barriers, including fence (i.e., use a straight fence without a bend). For protective screening, the total height of parapet and fence need not exceed 8'-0". The boom extension on most snooper trucks does not exceed 11'-0" so provision must be made to get the truck closer to the edge.

11. Where Traffic Railing is utilized between the roadway and an at grade sidewalk, early coordination with the roadway designer should occur to provide adequate clearances off of the structure to allow for proper safety hardware placement and sidewalk width. Additional clearance may be required in order to provide a crash cushion or other device to protect vehicles from the blunt end of the interior Traffic Railing off of the structure.

12. On shared-use bridges, fencing height and geometry shall be coordinated with the Region and the DNR (or other agencies) as applicable. Consideration shall be given to bridge use (i.e., multi-use/snowmobile may require vertical and horizontal clearances to allow grooming machine passage) and location (i.e., stream crossing vs. grade separation).

13. Per **LRFD [13.7.1.1]**, the use of raised sidewalks on structures shall be restricted to roadways with a design speed of 45 mph or less. The height of curbs for sidewalks is usually 6 inches. This height is more desirable than higher heights with regards to safety because it is less likely to vault vehicles. However, a raised curb is not considered part of the safety barrier system. On structure rehabilitations, the height of sidewalk may increase up to 8 inches to match the existing sidewalk height at the bridge approaches. Contact the Bureau of Structures Development Section if sidewalk heights in excess of 8 inches are desired. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for typical raised sidewalk detail information.

30.4 Railing Aesthetics

Railing aesthetics have become a key component to the design and delivery of bridge projects in Wisconsin. WisDOT Regions, local communities and their leaders use rail aesthetics to draw pedestrians to use the walkways on structures. With the increased desire to use, and frequency of use of aesthetics on railings, it has become increasingly important to set policy for railing aesthetics on bridge structures.

Railing aesthetics policies have been around for multiple decades. In the 1989 version of the AASHTO Standard Specifications, generalities were listed for use with designing bridge rails. Statements such as “Use smooth continuous barrier faces on the traffic side” and “Rail ends, posts, and sharp changes in the geometry of the railing shall be avoided to protect traffic from direction collision with the bridge rail ends” were used as policy and engineering judgment was required by each individual designer. This edition of the Standard Specifications aligned with NCHRP Report 350.

Caltrans conducted full-scale crash testing of various textured barriers in 2002. This testing was the first of its kind and produced acceptable railing aesthetics guidelines for single slope barriers for NCHRP Report 350 TL-3 conditions. Some of the allowable aesthetics were: sandblast textures with a maximum relief of 3/8”, geometric patterns inset into the face of the barrier 1” or less and featuring 45° or flatter chamfered or beveled edges, and any pattern or texture with a maximum relief of 2½” located 24” above the base of the barrier. Later in 2002, Harry W. Taylor, the Acting Director of the Office of Safety Design of FHWA, provided a letter to Caltrans stating that their recommendations were acceptable for use on all structure types.

In 2003, WisDOT published a paper titled, “Acceptable Community Sensitive Design Bridge Rails for Low Speed Streets & Highways in Wisconsin”. The goal of this paper was to streamline what railing aesthetics were acceptable for use on structures in Wisconsin. WisDOT policy at that time allowed vertical faced bridge rails in low speed applications to contain aesthetic modifications. For NHS structures, WisDOT allowed various types of texturing and relief based on crash testing and analysis. Ultimately, WisDOT followed many of the same requirements that were deemed acceptable by FHWA based on the Caltrans study in 2002.

NCHRP Report 554 – Aesthetic Concrete Barrier Design – was published in 2006 to (1) assemble a collection of examples of longitudinal traffic barriers exhibiting aesthetic characteristics, (2) develop design guidelines for aesthetic concrete roadway barriers, and (3) develop specific designs for see-through bridge rails. This publication serves as the latest design guide for aesthetic bridge barrier design and all bridge railings on structures in Wisconsin shall comply with the guidance therein.

<u>The application of aesthetics on bridge railings on structures in Wisconsin with a design speed exceeding 45 mph shall comply with the following guidance</u>:

1. All Traffic Railings shall meet the crash testing guidelines outlined in Section 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt

ends for impacts. Any raised feature that could serve as a blunt end or snag point shall be placed as follows:

Minimum of 2'-3" behind the front face toe of the railing when used with single slope parapets ("32SS", "36SS", "42SS", or "56SS").

Minimum of 2'-6" behind the front face toe of the railing when used with sloped face parapets ("LF" or "HF").

Minimum of 2'-0" behind the front face of the railing 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:

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 railing.

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.

30.5 Utilities

The maximum allowable conduits that can be placed in “SS”, “LF” or “HF” parapets are shown in the following sketches (“32SS” shown). Junction (Pull) boxes can only be used with 2 inch diameter conduit. The maximum length of 3 inch conduit is 190 feet, as no boxes are allowed.

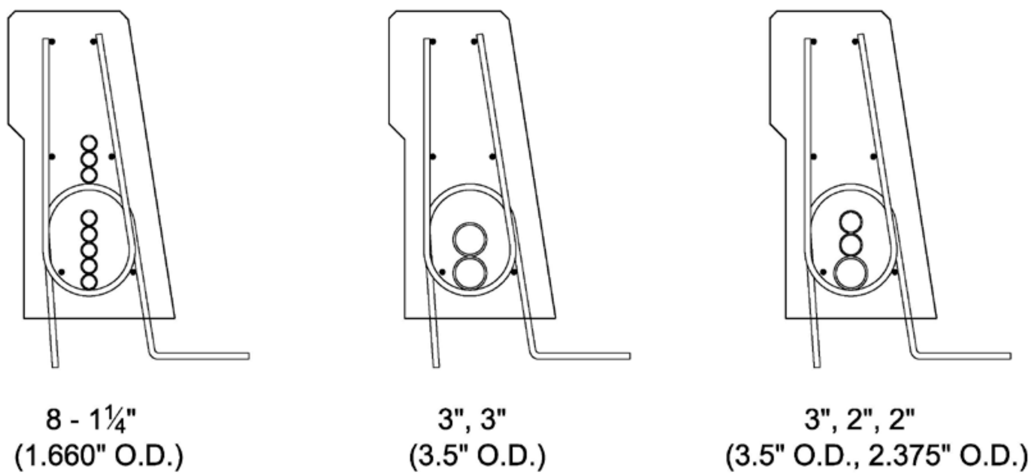


Figure 30.5-1
Maximum Allowable Conduits in “SS”, “LF” and “HF” Parapets

When light poles are mounted on top of parapets and the design speed exceeds 45 mph, the light pole must be located behind the back edge of the parapet. See Standard Detail 30.21 – Light Standard, Junction Box, & Expansion Fitting for “SS” Parapets – for typical light pole blister detail information. The poles should also be placed over the piers unless there is an expansion joint at that location. If an expansion joint is present, place 4 feet away. *FDM 9-25-5* addresses whether a bench mark disk should be set on a structure; however, structures are not usually preferred due to possible elevation changes from various causes. See Bridge Manual section 6.3.3.7 for more information regarding bench mark disks.

30.6 Protective Screening

Protective screening is a special type of fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a Traffic Railing (part of a Combination Railing) or on a sidewalk surface (Pedestrian Railing). The top of the protective screening may be bent inward toward the roadway, if mounted on a Traffic Railing and on a raised sidewalk, to prevent objects from being thrown off the overpass structure. The top of the protective screening may also be bent inward toward the sidewalk, if mounted directly to the deck when it is protected by a Traffic Railing between the roadway and a sidewalk at grade. Aesthetics are enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 30 and Chapter 37 Standard Details for protective screening detail information.

Examples of situations that warrant consideration of protective screening are:

1. Location with a history of, or instances of, objects being dropped or thrown from an existing overpass.

2. All new overpasses if there have been instances of objects being dropped or thrown at other existing overpasses in the area.

3. Overpasses near schools, playgrounds, residential areas or any other locations where the overpass may be used by children who are not accompanied by an adult.

In addition, all pedestrian overpasses should have protective screening on both sides.

When protective screening is warranted, the minimum design should require screening on the side of the structure with sidewalk. Designers can call for protective screening on sides without sidewalks if those sides are readily accessible to pedestrians.

Protective screening (or Pedestrian Railing) may be required for particular structures based on the safety requirements of the users on the structure and those below. Roadway designers, bridge designers, and project managers should coordinate this need and relay the information to communities involved when aesthetic details are being formalized.

Designers should ensure that where protective screening is called for, it does not interfere with sight distances between the overpass and any ramps connecting it with the road below. This is especially important on cloverleaf and partial cloverleaf type interchanges.

Protective screening is not always warranted. An example of when it may not be warranted is on an overpass without sidewalks where pedestrians do not have safe or convenient access to either side because of high traffic volumes and/or the number of traffic lanes that must be crossed.

Occasionally, access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

Fence repair should follow this same process except the damaged fencing would be removed and replaced with new fencing.

See Section 30.3 for additional guidance with regards to snooper truck access, screening height, and straight vs. bent fencing.

30.7 Medians

The typical height of any required median curb is 6 inches. This will prevent normal crossovers and reduce vaulting on low speed roadways without excessive dead load being applied to the superstructure. On structure rehabilitations, the height of median may increase up to 8" to match the existing median at the bridge approaches. Contact the Bureau of Structures Development Section if median heights in excess of 8 inches are desired. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for typical raised median detail information.

30.8 Railing Rehabilitation

The FHWA, in its implementation plan for MASH, requires that bridge railings on the NHS shall meet the requirements of MASH or NCHRP Report 350. In addition, FHWA states that "Agencies are encouraged to upgrade existing highway safety hardware that has not been accepted under MASH or NCHRP Report 350 during reconstruction projects, during 3R (Resurfacing, Restoration, Rehabilitation), or when the railing system is damaged beyond repair".

WisDOT requirements for the treatment of existing railings for various project classifications are outlined in Table 30.8-1:

Project Classification	**Railing Rehabilitation**
Preventative Maintenance (Resurfacing, Restoration)	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required. Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. <u>NHS Structures</u>: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350. <u>Non-NHS Structures</u>: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350.

3R (Resurfacing, Restoration, Rehabilitation)	If rehabilitation work, as part of the 3R project, is scheduled or performed which does not widen the structure nor affect the existing railing.	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required provided the minimum rail height requirement is met. (Minimum rail height shall be 27" for roadway design speed of 45 mph or less and 32" for roadway design speed exceeding 45 mph.) Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings (i.e., raised to meet the minimum rail height requirement) where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. <u>NHS Structures</u>: Existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement shall be upgraded to comply with MASH or NCHRP Report 350. <u>Non-NHS Structures</u>: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement be upgraded to comply with MASH or NCHRP Report 350.
	If rehabilitation work, as part of the 3R project, is scheduled or performed which widens the structure to either side, redecks (full-depth) any complete span of the structure, or if any work affecting the rail is done to the existing structure.	All railing on the structure must comply with MASH or NCHRP Report 350. Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.
4R (Resurfacing, Restoration, Rehabilitation, Reconstruction)		All railing on the structure must comply with MASH or NCHRP Report 350. Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.

<u>Table 30.8-1</u>
WisDOT Requirements for Retrofitting/Upgrading Bridge Railings to Current Standards

The minimum railing height shall be measured from the top inside face of the railing to the top of the roadway surface at the toe of railing.

For all railing rehabilitations that require upgrades to comply with MASH or NCHRP Report 350, railings shall be employed as discussed in Section 30.2.

The following is a list of typical railing types that are in service on structures in Wisconsin. The underlined railings comply with MASH, NCHRP Report 350, or NCHRP Report 230 and may remain in service within rehabilitation projects. The *italicized* railings do not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and shall be removed from service within rehabilitation projects.

1. Single slope parapet "32SS", "36SS", "42SS", "56SS". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 specifications and meet crash test criteria for TL-4.

2. Sloped face parapet "LF". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. Sloped face parapet "HF". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

4. Vertical face parapet "A". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic parapet "TX". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. Type "PF" tubular railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 and meets TL-2.

7. Type "H" railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

8. Timber Railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

9. Type "W" railing. Railing may be used for rehabilitation projects on non-NHS structures only. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 based on a May 1997 FHWA memorandum.

10. Type "M" railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 and meets TL-4

11. *Type "F" railing.* Railing may not be used for rehabilitation projects.

12. Sloped face parapet "B". Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230.

The region shall contact the Bureau of Structures Development Section to determine the sufficiency of existing railings not listed above.

Rehabilitation or improvement projects to historically significant bridges require special attention. Typically, if the original railing is present on a historic bridge, it will likely not meet current crash testing requirements. In some cases, the original railing will not meet current minimum height and opening requirements. There are generally two different options for upgrading railings on historically significant bridges – install a crash-tested Traffic Railing to the interior side of the existing railing and leave the existing railing in place or replace the existing railing with a crash-tested Traffic Railing. Other alternatives may be available but consultation with the Bureau of Structures Development Section is required.

30.9 Railing Guidance for Railroad Structures

Per an April 2013 memorandum written by M. Myint Lwin, Director of the FHWA Office of Bridge Technology, bridge parapets, railings, and fencing shall conform to the following requirements when used in the design and construction of grade separated highway structures over railroads:

1. For NHS bridges over railroad:

   Bridge railings shall comply with AASHTO standards. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   However, railings for use on NHS bridges over railroads shall be governed by the railroad's standards, regardless of whether the bridge is owned by the railroad or WisDOT. For the case where an NHS bridge crosses over railroads operated by multiple authorities with conflicting parapet, railing, or fencing requirements, standards as agreed by the various railroad authorities and as approved by WisDOT shall be used.

2. For non-NHS bridges over railroad:

   Bridge railings shall comply with the policies outlined within this chapter. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   All federally funded non-NHS bridges including those over railroads shall be governed by WisDOT's policies outlined above, even if they differ from the railroad's standards.

30.10 References

1. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications.*

2. American Association of State Highway and Transportation Officials. *Manual for Assessing Safety Hardware*.

3. National Cooperative Highway Research Program. *NCHRP Report 554 – Aesthetic Concrete Barrier Design*.

4. State of California, Department of Transportation. *Crash Testing of Various Textured Barriers*.

5. National Cooperative Highway Research Program. *NCHRP Report 350 – Recommended Procedures for the Safety Performance Evaluation of Highway Features*.

6. State of Wisconsin, Department of Transportation. *Facilities Development Manual*.

7. State of Wisconsin, Department of Transportation. *Wisconsin Bicycle Facility Design Handbook.*

Table of Contents

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.

36.3.4.3.2 Depth of Fill Less than 2.0 ft

Per **LRFD [5.14.5.3]**, for box culverts with less than 2.0 feet of fill follow **LRFD [5.8]** and **LRFD [5.13.3.6]**.

The shear resistance of the concrete, V_c, for slabs and walls of box culverts with less than 2.0 feet of fill, for one-way action per **LRFD [5.8.3.3]** shall be determined as:

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \leq 0.25f'_c\,b_v d_v$$

With variables defined above in 36.3.4.3.1.

For box culverts where the top slab is an integral part of the wearing surface (depth of fill equal zero) the top slab shall be checked for two-way action, as discussed in 18.3.3.2.2.

36.3.5 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**.

36.3.5.1 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

36.3.5.2 Crack Control Criteria

Per **LRFD [12.11.3]**, the provisions of **LRFD [5.7.3.4]** shall apply to crack width control in box culverts. All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r, specified in **LRFD [5.4.2.6]** for Service I Limit State. The spacing, s, (in inches) of mild steel reinforcement in the layer closest to the tension face shall satisfy:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(in.)}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

γ_e	=	Exposure factor (1.0 for Class 1 exposure condition, 0.75 for Class 2 exposure condition, see **LRFD [5.7.3.4]** for guidance)
d_c	=	Thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)
f_{ss}	=	Tensile stress in steel reinforcement at the service limit state (ksi) $\leq$ 0.6 f_y
h	=	Overall thickness or depth of the component (in.)

WisDOT Policy Item:

A class 1 exposure factor, γ_e = 1.0, shall be used for all cases for cast-in-place box culverts except for the top steel in the top slab of a box culvert with zero fill, where a class 2 exposure factor, γ_e = 0.75, shall be used.

36.3.6 Minimum Reinforcement Check

Per **LRFD [12.11.4.3]**, the area of reinforcement, A_s, in the box culvert cross-section should be checked for minimum reinforcement requirements per **LRFD [5.7.3.3.2]**.

The area of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, or moment capacity at least equal to the lesser of:

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

γ_1	=	1.6	flexural cracking variability factor
γ_3	=	0.67	ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement
f_r	=	$0.37\sqrt{f'_c}$	Modulus of rupture (ksi) **LRFD [5.4.2.6]**
I_g	=	Gross moment of inertia (in^4)	

factors and limit state combinations. Water pressure in culvert barrels is ignored. Refer to **LRFD [3.7.1]** for additional information regarding water pressure.

36.4.6 Live Loads (LL)

Live load consists of the standard AASHTO LRFD trucks and tandem. Per **LRFD [3.6.1.3.3]**, design loads are always axle loads (single wheel loads should not be considered) and the lane load is not used.

Where the depth of fill over the box is less than 2 feet, the wheel loads are distributed per **LRFD [4.6.2.10]**. Where the depth of fill is 2 feet or more, the wheel loads shall be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area **LRFD [3.6.1.2.5]**, increased by the live load distribution factor (LLDF) in **LRFD[Table 3.6.1.2.6a-1]**, using the provisions of **LRFD [3.6.1.2.6b-c]**. Where areas from distributed wheel loads overlap at the top of the culvert, the total load is considered as uniformly distributed over the rectangular area (A_{LL}) defined by the outside limits described in **LRFD [3.6.1.2.6b-c]**.

Per **LRFD [3.6.1.2.6a]**, for single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between inside faces of end walls.

Skew is not considered for design loads.

36.4.6.1 Depth of Fill Less than 2.0 ft.

Where the depth of fill is less than 2.0 ft, follow **LRFD [4.6.2.10]**.

36.4.6.1.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [4.6.2.10.2]**. Use a single lane and the single lane multiple presence factor of 1.2.

Distribution length perpendicular to the span:

$$E = (96 + 1.44(S))$$

Where:

E = Equivalent distribution width perpendicular to span (in.)

S = Clear span (ft)

The distribution of wheel loads perpendicular to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-2.

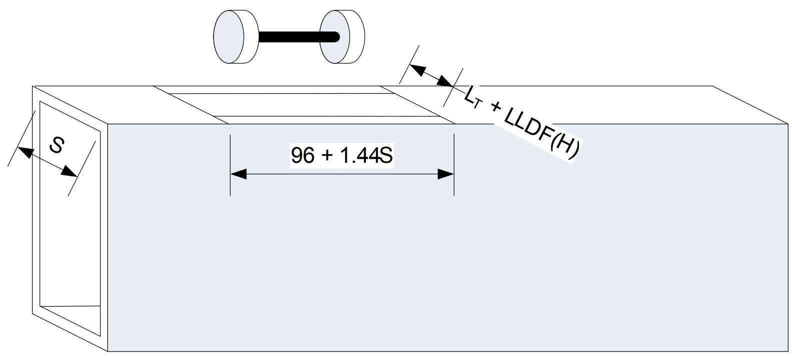


Figure 36.4-2
Distribution of Wheel Loads Perpendicular to Span, Depth of Fill Less than 2.0 feet

Distribution length parallel to the span:

$$E_{span} = (L_T + LLDF\ (H))$$

Where:

E_{span}	=	Equivalent distribution length parallel to span (in.)
L_T	=	Length of tire contact area parallel to span, as specified in **LRFD [3.6.1.2.5]** (in.)
LLDF	=	Factor for distribution of live load with depth of fill, 1.15, as specified in **LRFD [Table 3.6.1.2.6a-1]**.
H	=	Depth of fill from top of culvert to top of pavement (in.)

The distribution of wheel loads parallel to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-3.

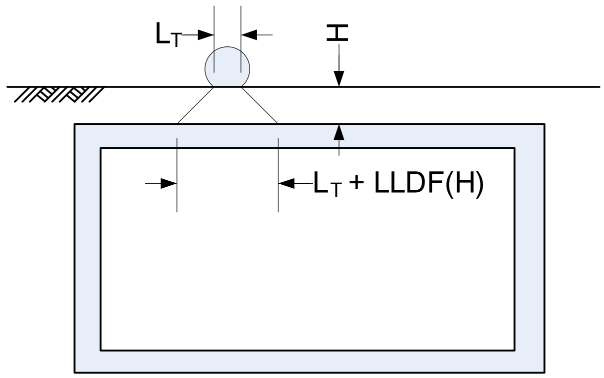


Figure 36.4-3
Distribution of Wheel Loads Parallel to Span, Depth of Fill Less than 2.0 feet

36.4.6.1.2 Case 2 - Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab using the equations specified in **LRFD [4.6.2.1]** for concrete decks with primary strips perpendicular to the direction of traffic per **LRFD[4.6.2.10.3]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

For a cast-in-place box culvert, the width of the primary strip, in inches is:

+M: 26.0 + (6.6)(S)

-M: 48.0 + (3.0)(S)

as stated in **LRFD [Table 4.6.2.1.3-1]**

Where:

S	=	Spacing of supporting components (ft)
+M	=	Positive moment
-M	=	Negative moment

36.4.6.2 Depth of Fill Greater than or Equal to 2.0 ft.

Where the depth of fill is 2.0 ft or greater, follow **LRFD [3.6.1.2.6b-c]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

36.4.6.2.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [3.6.1.2.6b]**.

For live load distribution transverse to span, the wheel/axle load interaction depth, H_{int-t}, shall be:

$$H_{int-t} = \frac{S_w - W_t/12 - 0.06D/12}{LLDF} \quad \text{(ft)}$$

where $H < H_{int-t}$ (no lateral interaction); then $W_w = W_t/12 + LLDF \cdot (H) + 0.06 \cdot (D/12)$

where $H \geq H_{int-t}$ (lateral interaction); then $W_w = W_t/12 + S_w + LLDF \cdot (H) + 0.06 \cdot (D/12)$

For live load distribution parallel to span, the wheel/axle load interaction depth H_{int-p} shall be:

$$H_{int-p} = \frac{S_a - \ell_t/12}{LLDF} \quad \text{(ft)}$$

where $H < H_{int-p}$ (no longit. interaction); then $\ell_w = \ell_t/12 + LLDF \cdot (H)$

where $H \geq H_{int-p}$ (longit. interaction); then $\ell_w = \ell_t/12 + S_a + LLDF \cdot (H)$

Where:

D	=	Clear span of the culvert (in)
H	=	Depth of fill from top of culvert to top of pavement (in)
H_{int-t}	=	Wheel interaction depth transverse to span (ft)
H_{int-p}	=	Axle interaction depth parallel to span (ft)
LLDF	=	Live load distribution factor per **LRFD [Table 3.6.1.2.6a-1]**; (1.15)
W_t	=	Width of tire contact area, per **LRFD [3.6.1.2.5]**; (20 in)
ℓ_t	=	Length of tire contact area, per **LRFD [3.6.1.2.5]**; (10 in)
S_w	=	Wheel spacing; (6.0 ft)

S_a = Axle spacing (ft)

W_w = Live load patch width at depth H (ft)

ℓ_w = Live load patch length at depth H (ft)

$A_{LL} = \ell_w \cdot W_w$

Where:

A_{LL} = Rectangular area at depth H (ft^2)

The live load vertical crown pressure shall be:

$$P_L = \frac{P(1 + IM/100)(m)}{A_{LL}}$$

Where:

IM = Dynamic load allowance (%); (see 36.4.8)

m = Multiple presence factor per **LRFD [3.6.1.1.2]**

P = Live load applied at surface on all interacting wheels (kip)

P_L = Live load vertical crown pressure (ksf)

The longitudinal and transverse distribution widths for depths of fill greater than or equal to 2.0 feet are illustrated in Figure 36.4-4.

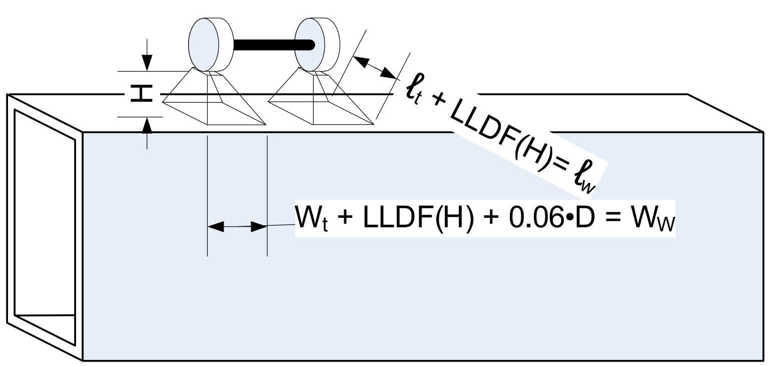


Figure 36.4-4
Distribution of Wheel Loads, Depth of Fill $\geq$ 2.0 feet (no lateral interaction)

36.4.6.2.2 Case 2 – Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab as described in **LRFD [3.6.1.2.6c]**.

36.4.7 Live Load Soil Pressures

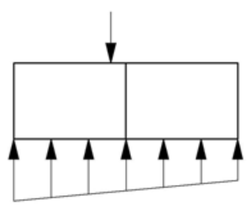


Figure 36.4-5
Vertical Soil Pressure under Culvert

The soil pressure on the bottom of the box is determined by moving the live load across the box. Find the location where the live load causes the maximum effects on the top slab of the box. At that location, determine the soil pressure diagram that will keep the system in equilibrium. Use the effects of this soil pressure in the bottom slab analysis.

36.4.8 Dynamic Load Allowance

Dynamic load allowance decreases as the depth of fill increases. **LRFD [3.6.2.2]** states that the impact on buried components shall be calculated as:

$$IM = 33(1.0 - 0.125(D_E)) \geq 0\%$$

Where:

D_E = Minimum depth of earth cover above the structure (ft)

36.4.9 Location for Maximum Moment

Create influence lines and use notional loading to determine the location for maximum moment. In this analysis, include cases for variable axle spacing and reverse axle order for unsymmetrical loading conditions.

For notional vehicles, only the portion of the loading that contributes to the effect being maximized is included. This is illustrated in Figure 36.4-6.

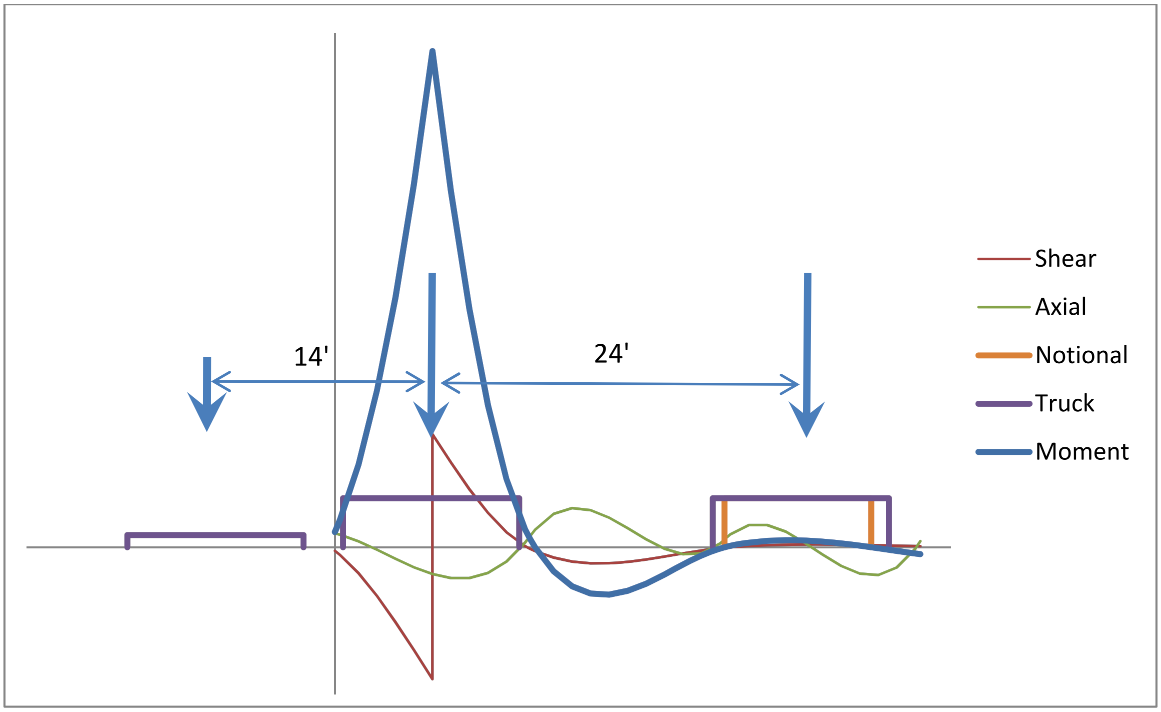


Figure 36.4-6
Application of Notional Loading using Influence Lines

The maximum positive moment results when the middle axial load is centered at the first positive peak while the variable rear axial spacing is 24 feet. Only the portion of the rear axial load in the positive region of the moment influence line is considered. The middle axial load and the portion of the rear axial in the positive region of the moment influence line are loaded on the shear and axial influence lines to compute the corresponding effects. Both positive and negative portions of the shear and axial influence lines are used when computing the corresponding effects. This process is repeated for maximizing the negative moment, shear and axial effects and computing the corresponding effects.

36.5 Design Information

Sidesway of the box is not considered because of the lateral support of the soil.

The centerline of the walls and top and bottom slabs are used for computing section properties and dimensions for analysis.

WisDOT Policy Item:

For skews 20 degrees or less, place the reinforcing steel along the skew. For skews over 20 degrees, place the reinforcing steel perpendicular to the centerline of box.

Culverts are analyzed as if the reinforcing steel is perpendicular to the centerline of box for all skew angles.

The minimum thickness of the top and bottom slab is 6½ inches. Minimum wall thickness is based on the inside opening of the box (height) and the height of the apron wall above the floor. Use the following table to select the minimum wall thickness that meets or exceeds the three criteria in the table.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

All slab thicknesses are rounded to the next largest ½ inch.

Top and bottom slab thicknesses are determined by shear and moment requirements. Slab thickness shall be adequate to carry the factored shear without shear reinforcement.

All bar steel is detailed as being 2 inches clear with the following exceptions:

- The bottom steel in the bottom slab is detailed with 3 inches clear
- The top steel in the top slab of a box culvert with no fill is detailed with 2½ inches clear

A haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Only 45º haunches shall be used. Minimum haunch depth and length is 6 inches. Haunch dimensions are increased in 3 inch increments.

The slab thickness required is determined by moment or shear, whichever governs.

The shear in the top and bottom slabs is assumed to occur at a distance "d" from the face of the walls. The value for "d" equals the distance from the centroid of the reinforcing steel to the face of the concrete in compression. When a haunch is used, shear must also be checked at the end of the haunch.

For multi-cell culverts make interior and exterior walls of equal thickness.

Culverts shall be designed for the range of fill between the shoulders of the roadway. To accommodate future widening of the roadway, reduced sections may not be used on the ends of the culvert where there is less fill. Exceptions may be made with the approval of the Bureau of Structures where the culvert has high fills and a reduced section could be used for at least two panel pours per end of culvert. Culvert extensions shall be designed for the same range of fills as the original culvert. The extension design shall not have lower capacity than the original culvert. Maximum length of panel pour is 40 feet.

Barrel lengths are based on the roadway sections and wing lengths are based on a minimum 2 1/2:1 slope of fill from the top of box to apron. Consideration shall be given to match the typical roadway cross slope.

Dimensions on drawings are given to the nearest ¼ inch only.

36.6 Detailing of Reinforcing Steel

To calculate the required bar steel area and cutoff points a maximum positive and negative moment envelope is computed. It is assumed that the required bar lengths in the top slab are longer than those in the bottom slab. Therefore, cutoff points are computed for the top slab and are also used in the bottom slab.

36.6.1 Bar Cutoffs

Per **LRFD [5.11.1.2.1]**, all flexural reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

- The effective depth of the member
- 15 times the nominal diameter of the bar
- 1/20 of the clear span

Continuing reinforcement shall extend not less than the development length, ℓ_d (**LRFD [5.11.2]**) beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

Per **LRFD [5.11.1.2.2]**, at least one-third of the positive moment reinforcement in simple span members and one-fourth of the positive moment reinforcement in continuous span members shall extend along the same face of the member beyond the centerline of the support. In beams, such extension shall not be less than 6.0 in.

Per **LRFD [5.11.1.2.3]**, at least one-third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than:

- The effective depth of the member
- 12 times the nominal diameter of the bar
- 0.0625 times the clear span

36.6.2 Corner Steel

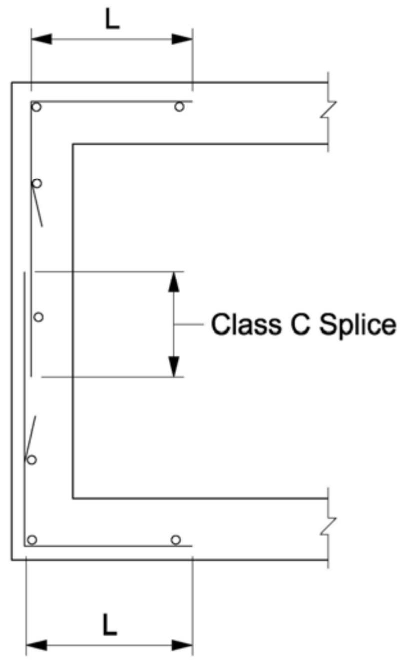


Figure 36.6-1
Layout of Corner Steel

The area of steel required is the maximum computed from using the top and bottom corner moments and the thickness of the slab or wall, whichever controls. Identical bars are used in the top and bottom corners. Identical length bars are used in the left and right corners if the bar lengths are within 2 feet of one another. Top and bottom negative steel is cut in the walls and detailed in two alternating lengths when a savings of over 2 feet in a single bar length can be obtained. Corner steel is always lapped at the center of the wall. If two bar lengths are used, only alternate bars are lapped.

Distance "L" is computed from the maximum negative moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1.

36.6.3 Positive Moment Slab Steel

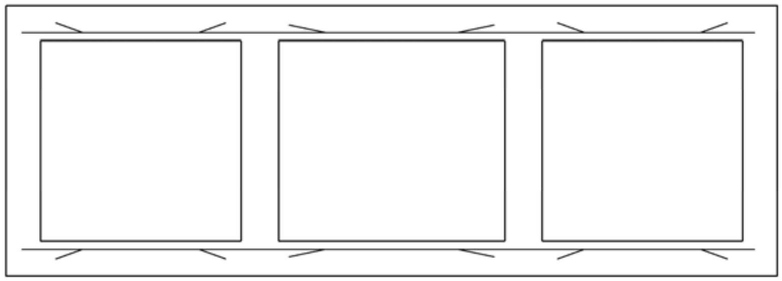


Figure 36.6-2
Layout of Positive Moment Steel

The area of steel required is determined by the maximum positive moments in each span. Top and bottom slab reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

When two alternating bar lengths are detailed in multi-cell culverts, run every other positive bar across the entire width of box. If this requires a length longer than 40 feet, lap them over an interior wall. For 2 or more cells, if the distance between positive bars of adjacent cells is 1 foot or less, make the bar continuous.

The cutoff points of alternate bars are determined from the maximum positive moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1. These same points are used in the bottom slab. Identical bar lengths are used over multiple cells if bars are within 2 feet of one another.

36.6.4 Negative Moment Slab Steel over Interior Walls

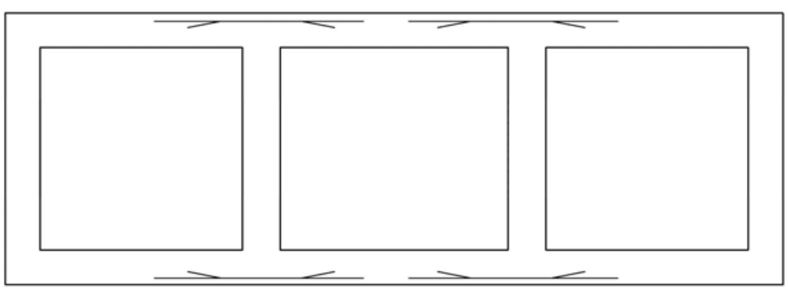


Figure 36.6-3
Layout of Negative Moment Steel

If no haunch is present, the area of steel required is determined by using the moment and effective depth at the face of the interior wall. If the slab is haunched, the negative reinforcement is determined per **LRFD [12.11.4.2],** which states that the negative moment is determined at the intersection of the haunch and uniform depth member. Top and bottom slab reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

Cutoff points are determined from the maximum negative moment envelope of the top slab and shall include the extension lengths discussed in 36.6.1. The same bar lengths are then used in the bottom slab. Identical bar lengths are used over multiple interior walls if bars are within 2 feet of one another. The minimum length of any bar is 2 times the development length. For culverts of 3 or more cells, if the clear distance between negative bars of adjacent spans is 1 foot or less, make the bar continuous across the interior spans.

When there is no fill over the top slab, run the negative moment reinforcing steel across the entire width of the culvert. Refer to 36.6.8 for temperature and shrinkage requirements.

36.6.5 Exterior Wall Positive Moment Steel

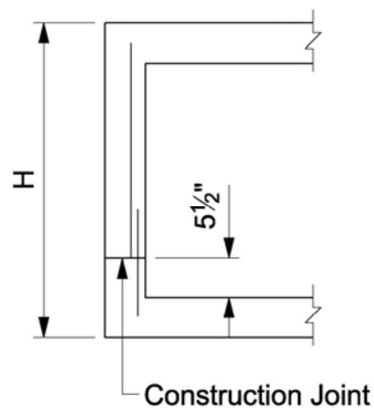


Figure 36.6-4
Layout of Exterior Wall Steel

The area of steel is determined by the maximum positive moment in the wall. A minimum of #4 bars at 18 inches is supplied. The wall bar is extended to 2 inch top clear and the dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed.

36.6.6 Interior Wall Moment Steel

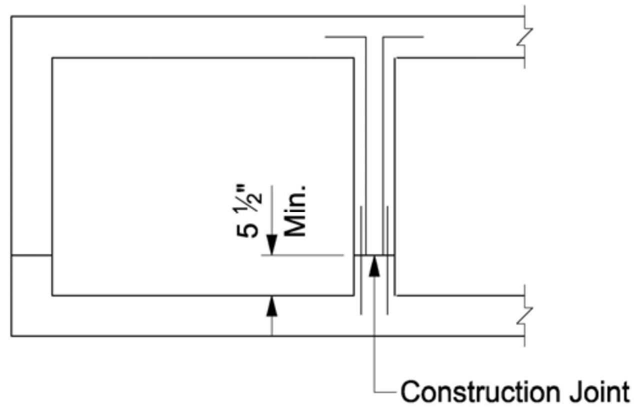


Figure 36.6-5
Layout of Interior Wall Steel

The area of steel is determined from the maximum moment at the top of the wall and the effective wall thickness. A minimum of #4 bars at 18 inches is supplied. Identical steel is provided at both faces of the wall. A 1 foot, 90 degree bend, is provided in the top slab with the horizontal portion being just below the negative moment steel. The dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed. When a haunch is provided, the construction joint is placed a distance above the bottom slab equal to the haunch depth plus 2 inches.

36.6.7 Distribution Reinforcement

Per **LRFD [5.14.4.1]**, transverse distribution reinforcement is not required for culverts where the depth of fill exceeds 2.0 feet.

Per **LRFD [12.11.2.1]**, provide distribution reinforcement for culverts with less than or equal to 2 feet of fill in accordance with **LRFD [9.7.3.2]**, which states that reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows (for primary reinforcement parallel to traffic):

$$\text{Percentage} = \frac{100}{\sqrt{S}} \leq 50\%$$

Where:

S = Effective span length (ft) (for slabs monolithic with walls, this distance is taken as the face-to-face distance per **LRFD [9.7.2.3]**)

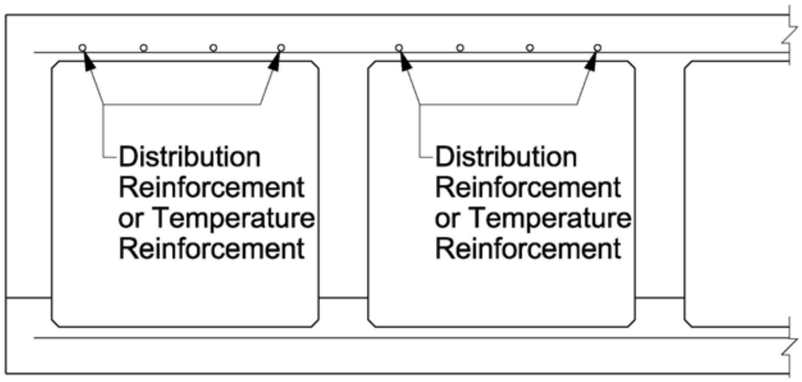


Figure 36.6-6
Layout of Distribution Steel

36.6.8 Temperature Reinforcement

Temperature reinforcement is required on all wall and slab faces in each direction that does not already have strength or distribution reinforcement. Per **LRFD [12.11.4.3.1]**, provide shrinkage and temperature reinforcement in walls and slabs in accordance with **LRFD [5.10.8]**, which states that the area of shrinkage and temperature steel per foot on each face and in each direction shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$

$$0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	Area of reinforcement in each direction and each face (in^2/ft)
b	=	Least width of component section (in.)
h	=	Least thickness of component section (in.)
f_y	=	Specified yield strength of reinforcing bars ≤ 75 (ksi)

Where the least dimension varies along the length of the component, multiple sections should be examined to represent the average condition at each section.

Temperature steel is always #4 bars at a maximum spacing of 18 inches. When the top slab has no fill on top use a minimum of #4 bars at 12 inch centers in both directions in the top of the top slab.

36.7 Box Culvert Aprons

Five types of box culvert aprons are used. They are referred to as Type A, B, C, D and E. The angle that the wings make with the direction of stream flow is the main difference between the five types. The allowable headwater and other hydraulic requirements are what usually determine the type of apron required. Physical characteristics at the site may also dictate a certain type. For hydraulic design of different apron types see Chapter 8.

36.7.1 Type A

Type A, because of its poor hydraulic properties, is generally not used except for cattle or pedestrian underpasses.

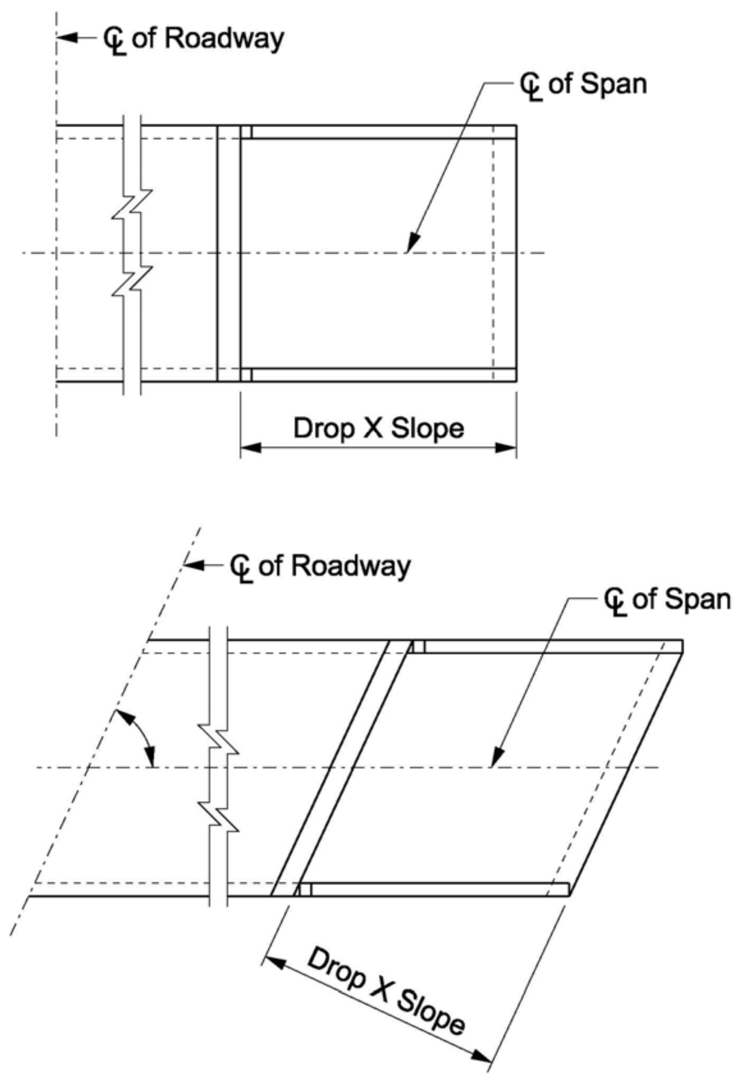
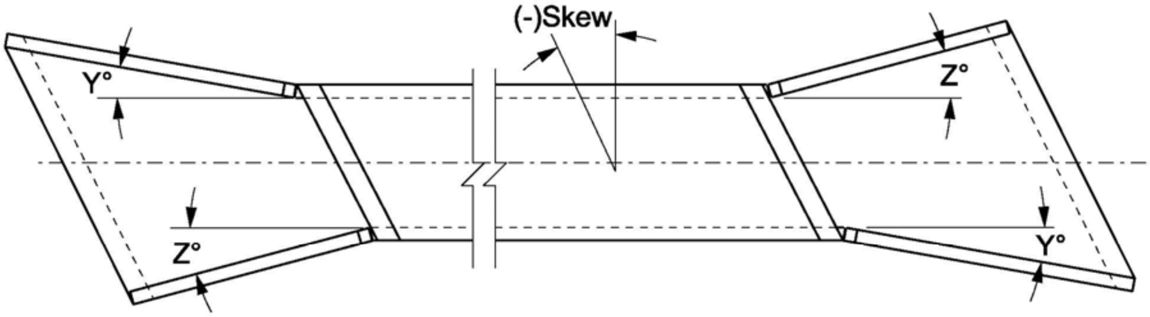


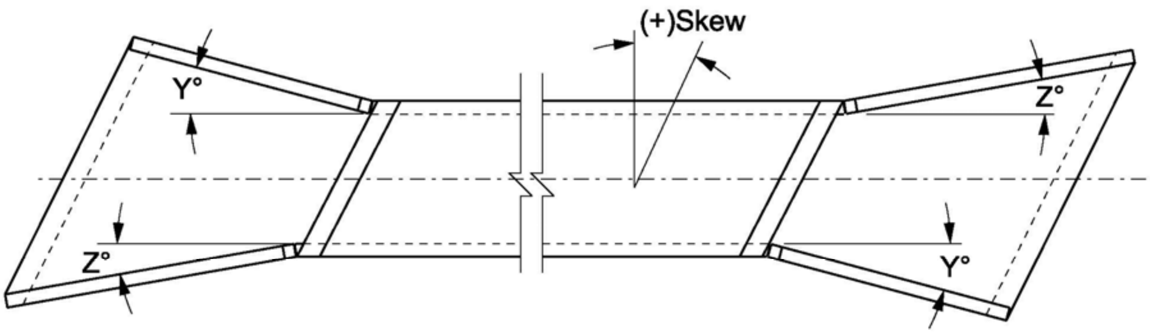
Figure 36.7-1
Plan View of Type A

36.7.2 Type B, C, D

Type B is used for outlets. Type C & D are of equal efficiency but Type C is used most frequently. Type D is used for inlets when the water is entering the culvert at a very abrupt angle. See Figure 36.7-2 for Wing Type B, C and D for guidance on wing angles for culvert skews.



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	25°	30°	40°	45°
15.0°	22.5°	10°	15°	25°	30°	35°	45°
22.5°	37.5°	10°	15°	20°	30°	30°	45°
37.5°	45.0°	10°	15°	15°	30°	25°	45°
45.0°	52.5°	5°	15°	15°	30°	20°	45°
52.5°	67.5°	5°	15°	10°	30°	15°	45°
67.5°	75.0°	5°	15°	5°	30°	10°	45°
75.0°	82.5°	0°	15°	5°	30°	5°	45°
82.5°	90.0°	0°	15°	0°	30°	0°	45°



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	30°	25°	45°	40°
15.0°	22.5°	15°	10°	30°	25°	45°	35°
22.5°	37.5°	15°	10°	30°	20°	45°	30°
37.5°	45.0°	15°	10°	30°	15°	45°	25°
45.0°	52.5°	15°	5°	30°	15°	45°	20°
52.5°	67.5°	15°	5°	30°	10°	45°	15°
67.5°	75.0°	15°	5°	30°	5°	45°	10°
75.0°	82.5°	15°	0°	30°	5°	45°	5°
82.5°	90.0°	15°	0°	30°	0°	45°	0°

Figure 36.7-2
Wing Type B, C, D (Angles vs. Skew)

36.7.3 Type E

Type E is used primarily in urban areas where a sidewalk runs over the culvert and it is necessary to have a parapet and railing along the sidewalk. For Type E the wingwalls run parallel to the roadway just like the abutment wingwalls of most bridges. It is also used where Right of Way (R/W) is a problem and the aprons would extend beyond the R/W for other types. Wingwall lengths for Type E wings are based on a minimum channel side slope of 1.5 to 1.

36.7.4 Wingwall Design

Culvert wingwalls are designed using a 1 foot surcharge height, a unit weight of backfill of 0.120 kcf and a coefficient of lateral earth pressure of 0.5, as discussed in 36.4.3. When the wingwalls are parallel to the direction of traffic and where vehicular loads are within ½ the wall height from the back face of the wall, design using a surcharge height representing vehicular load per **LRFD [Table 3.11.6.4-2]**. Load and Resistance Factor Design is used, and the load factor for lateral earth pressure of γ_{EH} = 1.69 is used, based on past design experience. The lateral earth pressure was conservatively selected to keep wingwall deflection and cracking to acceptable levels. Many wingwalls that were designed for lower horizontal pressures have experienced excessive deflections and cracking at the footing. This may expose the bar steel to the water that flows through the culvert and if the water is of a corrosive nature, corrosion of the bar steel will occur. This phenomena has lead to complete failure of some wingwalls throughout the State.

For wing heights of 7 feet or less determine the area of steel required by using the maximum wall height and use the same bar size and spacing along the entire wingwall length. The minimum amount of steel used is #4 bars at 12 inch spacing. Wingwall thickness is made equal to the barrel wall thickness.

For wing heights over 7 feet the wall length is divided into two or more segments and the area of steel is determined by using the maximum height of each segment. Use the same bar size and spacing in each segment.

Wingwalls must satisfy Strength I Limit State for flexure and shear, and Service I Limit State for crack control, minimum reinforcement, and reinforcement spacing. Adequate shrinkage and temperature reinforcement shall be provided.

36.8 Box Culvert Camber

Camber of culverts is a design compensation for anticipated settlement of foundation soil beneath the culvert. Responsibility for the recommendation and calculation of camber belongs to the Regional Soils Engineer. Severe settlement problems with accompanying large camber are to be checked with the Geotechnical Section.

Both total and differential settlement need to be considered to determine the amount of box camber required to avoid adverse profile sag and undesirable separation at culvert joints per **LRFD[12.6.2.2]**. If the estimated settlement is excessive, contingency measures will need to be considered, such as preloading with embankment surcharge, undercutting and subgrade stabilization. To evaluate differential settlement, it will be necessary to calculate settlement at more than one point along the length of the box culvert.

36.8.1 Computation of Settlement

Settlement should be evaluated at the Service Limit state in accordance with **LRFD [12.6.2.2]** and **LRFD [10.6.2]**, and consider instantaneous elastic consolidation and secondary components. Elastic settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. Consolidation settlement is the gradual compression of the soil skeleton when excess pore pressure is forced out of the voids in the soil. Secondary settlement, or creep, occurs as a result of plastic deformation of the soil skeleton under constant effective stress. Secondary settlement is typically not significant for box culvert design, except where there is an increase in effective stress within organic soil, such as peat. If secondary settlement is a concern, it should be estimated in accordance with **LRFD [10.6.2.4]**.

Total settlement, including elastic, consolidation and secondary components may be taken in accordance with **LRFD [10.6.2.4.1]** as:

$$S_t = S_e + S_c + S_s$$

Where:

S_t = Total settlement (ft)

S_e = Elastic settlement (ft)

S_c = Primary consolidation settlement (ft)

S_s = Secondary settlement (ft)

To compute settlement, the subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about 3 times the box width. The maximum layer thickness should be 10 feet.

Primary consolidation settlement for normally-consolidated soil is computed using the following equation in accordance with **LRFD [10.6.2.4.3]**:

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right]$$

Where:

S_c = Primary consolidation settlement (ft)

H_c = Initial height of compressible soil layer (ft)

e_o = Void ratio at initial vertical effective stress

C_c = Compression index which is a measure of the compressibility of a soil. It is the slope of the straight-line part of the e-log p curve from a conventional consolidation (oedometer) test.

σ'_f = Final vertical effective stress at midpoint of soil layer under consideration (ksf)

σ'_o = Initial vertical effective stress at midpoint of soil layer under consideration (ksf)

If the soil is overconsolidated, reference is made to **LRFD [10.6.2.4.3]** to estimate consolidation settlement.

Further description for the above equations and consolidation test can be found in most textbooks on soil mechanics.

For preliminary investigations C_c can be determined from the following approximate formula, found in most soil mechanics textbooks:

Non organic soils: $C_c = 0.007\ (LL-10)$

Where:

LL = Liquid limit expressed as whole number.

If the in-place moisture content approaches the plastic limit the computed C_c is decreased by 75%. If the in-place moisture content is near the liquid limit use the computed value. If the in-place moisture content is twice the liquid limit the computed C_c is increased by 75%. For intermediate moisture contents the percent change to the computed C_c is determined from a straight line interpolation between the corrections mentioned above.

If settlements computed by using the approximate value of C_c exceed 1.5 feet, a consolidation test is performed. As in-place moisture content approaches twice the liquid limit, settlement is caused by a local shear failure and the consolidation equation is no longer applicable.

The consolidation equation is applied to only compressible silts and clays. Sands are of a lower compressibility and no culvert camber is required until the fill exceeds 25 feet. When the fill exceeds 25 feet for sand, a camber of 0.01 feet per foot of fill is used.

36.8.2 Configuration of Camber

The following guides are to be followed when detailing camber.

- It is unnecessary to provide gradual camber. "Brokenback" camber is closer to the actual settlement which occurs.
- Settlement is almost constant from shoulder point to shoulder point. It then reduces to the ends of the culvert at the edge of the fill.
- The ends of the culvert tend to come up if side slopes are steeper than 2½ to 1. With 2 to 1 side slopes camber is increased 10% to compensate for this rise.

36.8.3 Numerical Example of Settlement Computation

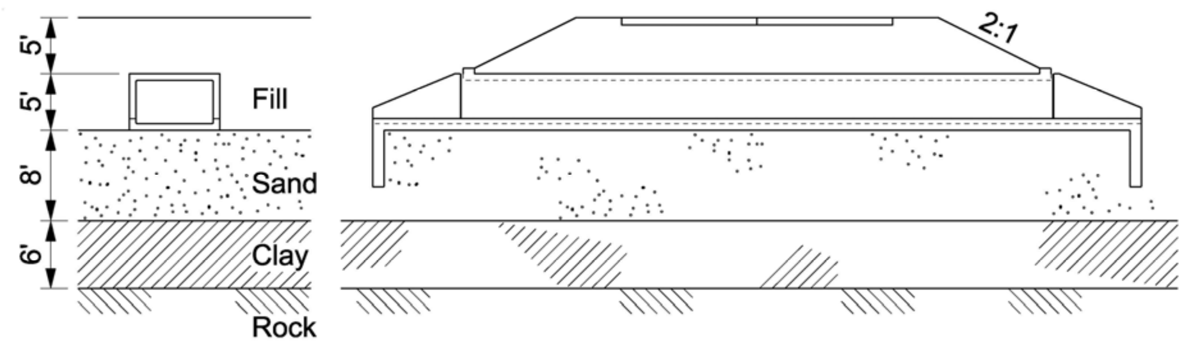


Figure 36.8-1
Soil Strata under Culvert

A box culvert rests on original ground consisting of 8 feet of sand and 6 feet of clay over bedrock. Estimate the settlement of the culvert if 10 feet of fill is placed on the original ground after the culvert is constructed. The in-place moisture content and liquid limit equal 40%. The initial void ratio equals 0.98. The unit weight of the clay is 105 pcf and that of the fill and sand is 110 pcf. There is no water table.

σ'_o = (8 ft)(110 pcf) + (3 ft)(105 pcf) = 1195 psf

$\sigma'_f = \sigma'_o$ + (10 ft)(110 pcf) = 1195 psf + 1100 psf = 2295 psf

C_c = 0.007 (40-10) = 0.21 (approximate value)

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right] = \frac{6\text{ft}}{1+0.98} 0.21 * \log_{10}\left[\frac{2295\text{psf}}{1195\text{psf}}\right] = 0.18\text{ft}$$

36.9 Box Culvert Structural Excavation and Structure Backfill

All excavations for culverts and aprons, unless on bedrock or fill, are undercut a depth of 6 inches. The upper limit of excavation is the existing ground line.

All spaces excavated and not occupied by the new structure are backfilled with structure backfill to the elevation and section existing prior to excavation within the length of the box. The backfill is placed to help eliminate settling problems on culverts. Backfill is placed in the undercut area under the apron. Usually 6 inches of structural backfill is placed under all boxes for construction purposes, which is covered by specification.

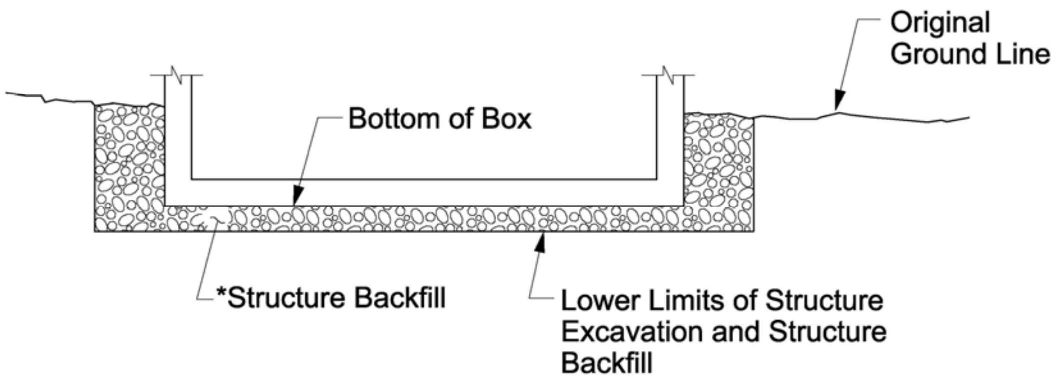


Figure 36.9-1
Limits for Excavation and Backfill

* Structure Backfill, No. 2 Washed Stone or Breaker Run Stone may be used to support culverts.

No backfill is placed under the box for culverts built on fills. The purpose of the backfill is to provide a solid base to pour the bottom slab. It is assumed that fill material provides this base without the addition of backfill.

36.10 Box Culvert Headers

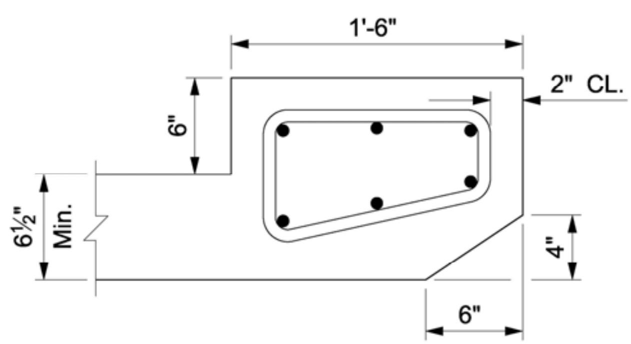
For skews of 20 degrees and less the main reinforcing steel is parallel to the end of the barrel. A header is not required for structural purposes but is used to prevent the fill material from spilling into the apron. A 12 inch wide by 6 inch high (above the top of top slab) header with nominal steel is therefore used for skews of 20 degrees and less on the top slab. No header is used on the bottom slab.

For skews over 20 degrees the main reinforcing is not parallel to the end of the barrel. The positive reinforcing steel terminates in the header and thus the header must support, in addition to its own dead load, an additional load from the dead load of the slab and fill above it. A portion of the live load may also have to be supported by the header.

The calculation of the actual load that a header must support becomes a highly indeterminate problem. For this reason a rational approach is used to determine the amount of reinforcement required in the headers. The design moment capacity of the header must be equal to or greater than 1.25 times the header dead load moment (based on simple span) plus 1.75 times a live load moment from a 16 kip load assuming 0.5 fixity at ends.

To prevent a traffic hazard, culvert headers are designed not to protrude above the ground line. For this reason the height of the header above the top of the top slab is typically selected to be 6 inches. The width of the header is standardized at 18 inches.

The header in the following figure gives the design moment capacities listed using d = 8.5 inches.



Bar Size	Moment Capacity ft.-kip
#6	46.1
#7	60.7
#8	76.5
#9	92.1
#10	109.1

Figure 36.10-1
Header Details (Skews > 20°)

The following size bars are recommended for the listed header lengths where "Header Length" equals the distance between C/L of walls in one cell measured along the skew.

Header Length	Bar Size [1]
To 11’	#7
Over 11’ to 14’	#8
Over 14’ to 17’	#9
Over 17’ to 20’	#10

Table 36.10-1
Header Reinforcement

[1] Use the bar size listed in each header and place 3 bars on the top and 3 bars on the bottom. Use a header on both the top and bottom slab. See the Standard *Box Culvert Details* in Chapter 36.

Where headers greater than 6 inches in height are used to retain roadway fill, the top slab shall be designed to handle the bending moment transmitted from the header. Additional reinforcement may be required.

Where barriers are placed on top of the culvert header, the barrier, header, and top slab shall be designed for vehicular impact forces.

36.11 Plan Detailing Issues

36.11.1 Weep Holes

Investigate the need for weep holes for culverts in cohesive soils. These holes are to relieve the hydrostatic pressure on the sides of the culverts. Where used, place the weep holes 1 foot above normal water elevation but a minimum of 1 foot above the lower sidewall construction joint. Do not place weep holes closer than 1 foot from the bottom of the top slab.

36.11.2 Cutoff Walls

Where dewatering the cutoff wall in sandy terrain is a problem, the concrete may be poured in the water. Place a note on the plans allowing concrete for the cutoff wall to be placed in the water.

36.11.3 Nameplate

Designate a location on the wingwall for placement of the nameplate. Locate nameplate on the first right wing traveling in the Cardinal direction (North/East).

36.11.4 Plans Policy

If a cast-in-place reinforced concrete box culvert is used, full plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

36.11.5 Rubberized Membrane Waterproofing

When required by the Standard Details, place the bid item "Rubberized Membrane Waterproofing" on the final plans. The quantity is given square yards.

36.12 Precast Four-Sided Box Culverts

In general, structural contractors prefer cast-in-place culverts while grading contractors prefer precast culverts. Precast culverts have been more expensive than cast-in-place culverts in the past, but allow for reduced construction time. Box culverts that are 4 feet wide by 6 feet high or less are considered roadway culverts. All other culverts require a B or C number along with the appropriate plans. All culverts requiring a number should be processed through the Bureau of Structures.

When a precast culvert is selected as the best structure type for a particular project during the design study phase, preliminary plans and complete detailed final plans are required to be sent to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

Sometimes a complete set of plans is created for a cast-in-place culvert and a precast culvert is stated to be an acceptable alternate. If the contractor selects the precast alternate, the contractor is to submit shop drawings, sealed by a professional engineer, to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

36.13 Three-Sided Structures

Three-sided box culvert structures are divided into two categories: cast-in-place three sided structures and precast three-sided structures. These structures shall follow the criteria outlined below.

36.13.1 Cast-In-Place Three-Sided Structures

To be developed

36.13.2 Precast Three-Sided Structures

Three-sided precast concrete structures offer a cost effective, convenient solution for a variety of bridge needs. The selection of whether a structure over a waterway should be a culvert, a three-sided precast concrete structure or a bridge is heavily influenced by the hydraulic opening. As the hydraulic opening becomes larger, the selection process for structure type progresses from culvert to three-sided precast concrete structure to bridge. Cost, future maintenance, profile grade, staging, skew, soil conditions and alignment are also important variables which should be considered. Culverts generally have low future maintenance; however, culverts should not be considered for waterways with a history or potential of debris to avoid channel cleanout maintenance. In these cases a three-sided precast concrete structure may be more appropriate. Three-sided precast concrete structures have the advantage of larger single and multiple openings, ease of construction, and low future maintenance costs.

A precast-concrete box culvert may be recommended by the Hydraulics Team. The side slope at the end or outcrop of a box culvert should be protected with guardrail or be located beyond the clear zone.

The hydraulic recommendations will include the Q_{100} elevation, the assumed flowline elevation, the required span, and the required waterway opening for all structure selections. The designer will determine the rise of the structure for all structure sections.

A cost comparison is required to justify a three-sided precast concrete structure compared to other bridge/culvert alternatives.

To facilitate the initiation of this type of project, the BOS is available to assist the Owners and Consultants in working out problems which may arise during plan development.

Some of the advantages of precast three-sided structures are listed below:

- Speed of Installation: Speed of installation is more dependent on excavation than product handling and placement. Precast concrete products arrive at the jobsite ready to install. Raw materials such as reinforcing steel and concrete do not need to be ordered, and no time is required on site to set up forms, place concrete, and wait for the concrete to cure. Precast concrete can be easily installed on-demand and immediately backfilled.

- Environmentally Friendly: Precast concrete is ready to be installed right off the delivery truck, which means less storage space needed for scaffolding and rebar. There is less noise pollution from ready-mix trucks continually pulling up on site and less waste as a result of using precast (i.e. no leftover steel, no pieces of scaffolding and no waste concrete piles). The natural bottom on a three-sided structure is advantageous to meet fish passage and DNR requirements.

- Quality Control: Because precast concrete products are produced in a quality-controlled environment with proper curing conditions, these products exhibit higher quality and uniformity over cast-in-place structures.

- Reduced Weather Dependency: Weather does not delay production of precast concrete as it can with cast-in-place concrete. Additionally, weather conditions at the jobsite do not significantly affect the schedule because the "window" of time required for installation is small compared to other construction methods, such as cast-in-place concrete.

- Maintenance: Single span precast three-sided structures are less susceptible to clogging from debris and sediment than multiple barrel culvers with equivalent hydraulic openings.

36.13.2.1 Precast Three-Sided Structure Span Lengths

WisDOT BOS allows and provides standard details for the following precast three-sided structure span lengths:

14'-0, 20'-0, 24'-0, 28'-0, 36'-0, 42'-0

Dimensions, rises, and additional guidance for each span length are provided in the standard details.

36.13.2.2 Segment Configuration and Skew

It is not necessary for the designer to determine the exact number and length of segments. The final structure length and segment configuration will be determined by the fabricator and may deviate from that implied by the plans.

A zero degree skew is preferable but skews may be accommodated in a variety of ways. Skew should be rounded to the nearer most-practical 5 deg., although the nearer 1 deg. is permissible where necessary. The range of skew is dependent on the design span and the fabrication limitations. Some systems are capable of fabricating a skewed segment up to a maximum of 45 degrees. Other systems accommodate skew by fabricating a special trapezoidal segment. If adequate right-of-way is available, skewed projects may be built with all right angle segments provided the angle of the wingwalls are adjusted accordingly. The designer shall consider the layout of the traffic lanes on staged construction projects when determining whether a particular three-sided precast concrete structure system is suitable.

Square segments are more economical if the structure is skewed. Laying out the structure with square segments will result in the greatest right-of-way requirement and thus allow ample space for potential redesign by the contractor, if necessary, to another segment configuration.

For a structure with a skew less than or equal to 15 deg., structure segments may be laid out square or skewed. Skewed segments are preferred for short structures (approximately less than 80 feet in length). Square segments are preferred for longer structures. However, skewed segments have a greater structural span. A structure with a skew of greater than 15 deg. requires additional analysis per the AASHTO LRFD Bridge Design Specifications. Skewed segments and the analysis both contribute to higher structure cost.

For a structure with a skew greater than 15 deg, structure segments should be laid out square. The preferred layout scheme for an arch-topped structure with a skew of greater than 15 deg should assume square segments with a sloping top of headwall to yield the shortest possible wingwalls. Where an arch-topped structure is laid out with skewed ends (headwalls parallel to the roadway), the skew will be developed within the end segments by varying the lengths of the legs as measured along the centerline of the structure. The maximum attainable skew is controlled by the difference between the full-segment leg length as recommended by the arch-topped-structure fabricator and a minimum leg length of 2 feet.

36.13.2.3 Minimum Fill Height

Minimum fill over a precast three-sided structure shall provide sufficient fill depth to allow adequate embedment for any required beam guard plus 6”. Refer to Standard 36.10 for further information.

Barriers mounted directly to the precast units are not allowed, as this connection has not been crash tested.

36.13.2.4 Rise

The maximum rises of individual segments are shown on the standard details. This limit is based on the fabrication forms and transportation. The maximum rise of the segment may also be limited by the combination of the skew involved because this affects transportation on the truck. Certain rise and skew combinations may still be possible but special permits may be required for transportation. The overall rise of the three-sided structure should not be a limitation when satisfying the opening requirements of the structure because the footing is permitted to extend above the ground to meet the bottom of the three-sided segment.

36.13.2.5 Deflections

Per **LRFD [2.5.2.6.2]**, the deflection limits for precast reinforced concrete three-sided structures shall be considered mandatory.

36.13.3 Plans Policy

If a precast or cast-in-place three-sided culvert is used, full design calculations and plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

The designer should use the span and rise for the structure selection shown on the plans as a reference for the information required on the title sheet. The structure type to be shown on the Title, Layout and General Plan sheets should be Precast Reinforced Concrete Three-Sided Structure.

The assumed elevations of the top of the footing and the base of the structure leg should be shown. For preliminary structure layout purposes, a 2-foot footing thickness should be assumed with the base of the structure leg seated 2 inches below the top-of-footing elevation. With the bottom of the footing placed at the minimum standard depth of 4 feet below the flow line elevation, the base of the structure leg should therefore be shown as 2'-2" below the flow line. An exception to the 4-foot depth will occur where the anticipated footing thickness is known to exceed 2 feet, where the footing must extend to rock, or where poor soil conditions and scour concerns dictate that the footing should be deeper.

The structure length and skew angle, and the skew, length and height of wingwalls should be shown. For a skewed structure, the wingwall geometrics should be determined for each wing. The sideslope used to determine the wing length should be shown on the plans.

If the height of the structure legs exceeds 10 feet, pedestals should be shown in the structure elevation view.

The following plan requirements shall be followed:

1. Preliminary plans are required for all projects utilizing a three-sided precast concrete structure.

2. Preliminary and Final plans for three-sided precast concrete structures shall identify the size (span x rise), length, and skew angle of the bridge.

3. Final plans shall include all geometric dimensions and a detailed design for the three-sided precast structure, all cast-in-place foundation units and cast-in-place or precast wingwalls and headwalls.

4. Final plans shall include the pay item Three-Sided Precast Concrete Structure and applicable pay items for the remainder of the substructure elements.

5. Final plans shall be submitted along with all pertinent special provisions to the BOS for review and approval.

In addition to foundation type, the wingwall type shall be provided on the preliminary and final plans. Similar to precast boxes, a wingwall design shall be provided which is supported independently from the three-sided structure. The restrictions on the use of cast-in-place or precast wings and headwalls shall be based on site conditions and the preferences of the Owner. These restrictions shall be noted on the preliminary and final plans.

36.13.4 Foundation Requirements

Precast and cast-in-place three-sided structures that are utilized in pedestrian or cattle underpasses can be supported on continuous spread or pile supported footings. Precast and cast-in-place three-sided structures that are utilized in waterway applications shall be supported on piling to prevent scour.

The footing should be kept level if possible. If the stream grade prohibits a level footing, the wingwall footings should be laid out to be constructed on the same plane as the structure footings. Continuity shall be established between the structural unit footing and the wingwall footing.

The allowable soil bearing pressure should be shown on the plans. Weak soil conditions could require pile foundations. If the footing is on piling, the nominal driving resistance should be shown. Where a pile footing is required, the type and size of pile and the required pile spacing, and which piles are to be battered, should be shown on the plans.

The geotechnical engineer should provide planning and design recommendations to determine the most cost effective and feasible foundation treatment to be used on the preliminary plans.

36.13.5 Precast Versus Cast-in-Place Wingwalls and Headwalls

The specifications for three-sided precast concrete structures permits the contractor to substitute cast-in-place for precast wingwalls and headwalls, and visa versa when cast-in-place is specified unless prohibited on the plans. Three-sided structures should be provided with adequate foundation support to satisfy the design assumptions permitting their relatively thin concrete section. These foundations are designed and provided in the plans. Spread footing foundations are most commonly used since they prove cost effective when rock or scour resistant soils are present with adequate bearing and sliding resistance. The use of precast spread footings shall be controlled by the planner and shall only be allowed when soil conditions permit and shall not be allowed to bear directly on rock or when rock is within 2 feet of the bottom of the proposed footing. When lower strength soils are present, or scour depths become large, a pile supported footing shall be used. The lateral loading design of the foundation is important because deflection of the pile or footing should not exceed the manufacturers' recommendations to preclude cracks developing.

36.14 Design Examples

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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 Sixth Edition - 2013 Interim)***

E36-1.1 Design Criteria

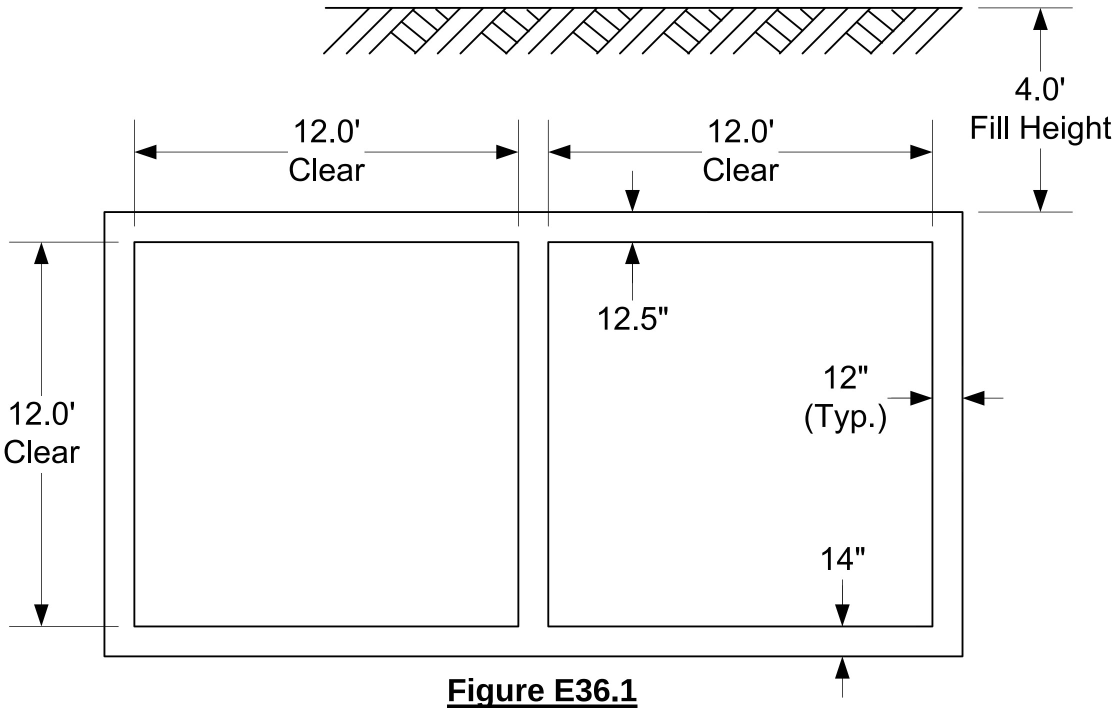

Figure E36.1
Box Culvert Dimensions

$NC = 2$	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

$$H_{apron} = 13.04$$

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 **[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 **[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:

$w_{sv} := \dfrac{W_E}{B_c}$ $\boxed{w_{sv} = 0.494}$

Horizontal Earth Load (EH):

soil horizontal earth load (magnitude at bottom and top of wall):

$k_o := 0.5$ coefficient of at rest lateral earth pressure **[36.4.3]**

$\gamma_s = 0.120$ unit weight of soil, kcf

$w_{sh\_bot} := k_o \cdot \gamma_s \cdot \left(Ht + \dfrac{t_{ts}}{12} + \dfrac{t_{bs}}{12} + H_s\right) \cdot 1$ $\boxed{w_{sh\_bot} = 1.09}$ 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:

$k_o = 0.5$ coefficient of lateral earth pressure

$\gamma_s = 0.120$ unit weight of soil, kcf

$LS_{ht} = 2.2$ live load surcharge height per **[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 <u>axle loads</u> (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.

Therefore, $mpf = 1.2$

Perpendicular to the span:

It is conservative to use the largest distribution factor from each span of the structure across the entire length of the culvert. Therefore, use the smallest span to calculate the smallest strip width. That strip width will provide the largest distribution factor.

$S := \min(W_1, W_2)$ clear span, ft $\boxed{S = 12.00}$ ft

The equivalent distribution width perpendicular to the span is:

$$E_{perp} := \frac{1}{12} \cdot (96 + 1.44 \cdot S) \qquad \boxed{E_{perp} = 9.44} \text{ ft}$$

Parallel to the span:

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

$L_T := 10$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$LLDF = 1.15$ live load distribution factor. From **LRFD [4.6.2.10.2]**, LLDF = 1.15 as specified in **LRFD [Table 3.6.1.2.6a-1]** for select granular backfill

The equivalent distribution width parallel to the span is:

$$E_{parallel} := \frac{1}{12} \cdot (L_T + LLDF \cdot H_s \cdot 12) \qquad \boxed{E_{parallel} = 5.43} \text{ ft}$$

The equivalent distribution widths parallel and perpendicular to the span create an area that the axial load shall be distributed over. The equivalent area is:

$$E_{area} := E_{perp} \cdot E_{parallel} \qquad \boxed{E_{area} = 51.29} \text{ ft}^2$$

For depths of fill 2.0 ft. or greater calculate the size of the rectangular area that the wheels are considered to be uniformly distributed over, per **[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:

$L_{eq\_i} := L_T + LLDF \cdot H_s \cdot 12$ $\boxed{L_{eq\_i} = 65.20}$ 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 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.

$LL1 = 0.94$ live load pressure (front wheel), psi

$LL2 = 3.78$ live load pressure (center wheel), psi

$LL3 = 3.78$ live load pressure (rear wheel), psi

E36-1.7 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.

$\boxed{\beta_1 = 0.85}$

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).

$$c := \frac{A_s \cdot f_s}{0.85 \cdot f'_c \cdot \beta_1 \cdot b}$$ $\boxed{c = 0.98}$ 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

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.7.3.2.3]**:

$a := \beta_1 \cdot c$ $\boxed{a = 0.83}$ in

$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right)\frac{1}{12}\right]$ $\boxed{M_n = 23.0}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 20.7}$ kip-ft

The required capacity:

Corner Moment $\boxed{Mstr1_{CB} = 17.3}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]**:

$b = 12.0$ in width of the concrete design section, in

$h = 12.0$ in height of the concrete design section, in

$f_r := 0.37 \cdot \sqrt{f'_c}$ modulus of rupture, ksi **LRFD [5.4.2.6]** $f_r = 0.69$ ksi

$I_g := \frac{1}{12} \cdot b \cdot h^3$ gross moment of inertia, in^4 $I_g = 1728.00$ in^4

$\frac{h}{2} = 6.0$ distance from the neutral axis to the extreme element

$S_c := \frac{I_g}{\frac{h}{2}}$ section modulus, in^3 $S_c = 288.00$ in^3

The corresponding cracking moment is:

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_c$ therefore, $M_{cr} = 1.1(f_r)S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement <u>for A615, Grade 60 reinforcement</u>

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 18.3}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CB} = 23.1}$ kip-ft

Is $M_r = 20.7$ kip-ft greater than the lesser of M_{cr} and 1.33*M_{str}? $\boxed{check = \text{"OK"}}$

Per **LRFD [5.7.3.4]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.0$ for Class 1 exposure condition

$h = 12.0$ height of the concrete design section, in

Calculate the ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face:

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.34}$$

Calculate the reinforcement ratio:

$$\rho := \frac{A_s}{b \cdot d_s} \qquad \boxed{\rho = 0.0043}$$

Calculate the modular ratio:

$$N := \frac{E_s}{E_c} \qquad \boxed{N = 8.06}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi). The moment arm used in the equation below to calculate f_{ss} is: (j) (h-d_c)

$$k := \sqrt{(\rho \cdot N)^2 + (2 \cdot \rho \cdot N)} - \rho \cdot N \qquad \boxed{k = 0.2301}$$

$$j := 1 - \frac{k}{3} \qquad \boxed{j = 0.9233}$$

$Ms1_{CB} = 11.18$ service moment, kip-ft

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \le 0.6\, f_y \qquad \boxed{f_{ss} = 30.23} \text{ ksi} \quad \le 0.6\, f_y \text{ O.K.}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

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- 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 less than 20 feet from centerline of track. Site circumstances will determine whether a crashwall is needed when the distance is between 20 and 25 feet.

- 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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40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapter 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

40.2 History

40.2.1 Concrete

Prior to 1975, all concrete structures were designed by the Service Load Method. The allowable design stress was 1400 psi based on an ultimate strength of 3500 psi except for deck slabs. In 1965, the allowable stress for deck slabs on stringers was reduced from 1400 to 1200 psi. The reason for the change was to obtain thicker concrete decks by using lower strength. The thicker deck was assumed to have more resistance to cracking and greater durability. During this time no changes were made in the physical properties of the concrete or the mixing quantities until 1974 when more cement was added to the mix for concrete used in bridge decks.

In 1980, the use of set retarding admixtures was required when the atmospheric temperature at the time of placing the concrete is 70°F or above; or when the atmospheric temperature is 50°F or above and it is expected that the time to place the concrete of any span or pour will require four or more hours. Retarding admixtures reduce concrete shrinkage cracking and surface spalling. Also, during the early 1980's a water reducing admixture was provided for Grades A and E concrete mixes to facilitate workability and placement.

40.2.2 Steel

Prior to 1975, Grade 40 bar steel was used in the design of reinforced concrete structures. This was used even though Grade 60 bars may have been furnished on the job. Allowable design stress was 20 ksi using the Service Load Method and 40 ksi using the Load Factor Method.

40.2.3 General

In 1978, Wisconsin Department of Transportation discovered major cracking on a two-girder, fracture critical structure, just four years after it was constructed. In 1980, on the same structure, major cracking was discovered in the tie girder flange of the tied arch span.

This is one example of the type of failures that transportation departments discovered on welded structures in the 1970's and '80's. The failures from welded details and pinned connections led to much stricter standards for present day designs.

All areas were affected: Design with identification of fatigue prone details and classifications of fatigue categories (AASHTO); Material requirements with emphasis on toughness and weldability, increased welding and fabrication standards with licensure of fabrication shops to minimum quality standards including personnel; and an increased effort on inspection of existing bridges, where critical details were overlooked or missed in the past.

Wisconsin Department of Transportation started an in-depth inspection program in 1985 and made it a full time program in 1987. This program included extensive non-destructive testing. Ultrasonic inspection has played a major role in this type of inspection. All fracture critical structures, pin and hanger systems, and pinned connections are inspected on a five-year cycle now.

40.3 Bridge Replacements

Bridge rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. In order to obtain federal funding eligibility for rehabilitation or replacement; the bridge must be Structurally Deficient or Functionally Obsolete. The Federal Sufficiency Number is a guide for federal participation which is required to be less than 50 for replacement. Also, Wisconsin DOT requires the Rate Score to be less than 75. Bridges are not eligible for replacement unless the Substructure or Superstructure Condition is 4 or less or the Inventory Rating is less than HS10 or the Alignment Appraisal is 4 or less.

A bridge becomes Structurally Deficient when the condition of the deck, superstructure or substructure is rated 4 or less; or when the inventory load capacity is less than 10 tons (89.0 kN); or when the waterway adequacy is rated a 2.

A bridge becomes Functionally Obsolete when the bridge roadway width, vertical clearance, or approach alignment is substandard (appraisal rating of 3 or less), or when the inventory load capacity is less than 15 tons; or when the waterway adequacy is rated a 3 or less.

Wisconsin DOT has established minimum roadway widths for bridges to remain in place on Rural, State and County Trunk Highways. As a minimum, bridge replacement is required for all bridges less than 100 ft. long and the useable width of the bridge is less than the following:

Design ADT	Rural Arterial Typically STH	Rural Arterial Typically CTH	Town Road
0-250	22’	20’	18’
251-750	22’	20’	22’
751-2000	Traveled Way +2’	Traveled Way + 2’	Traveled Way + 4’
2001-4000	Traveled Way + 4’	Traveled Way + 4’	Traveled Way + 4’
Over 4001	Traveled Way + 6’	Traveled Way + 6’	Traveled Way + 4’

Table 40.3-1
Bridge Clear Width Rehabilitation Requirements

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of insuring some level of acceptable serviceability. The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are sufficient to safely carry present and projected traffic. Information necessary to determine structure sufficiency includes structure inspection, inventory, traffic, maintenance, capacity and functional adequacy. The methods of reconstruction are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to insure that rehabilitation will remove all structural deficiencies. FHWA requires this review and Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation. On high cost bridges, a closer check of the Functionally Obsolete Criteria may be required. On high cost bridges a 2' shoulder is acceptable on a low speed, low volume roadway having a good accident record. After rehabilitation work is completed, the bridge should not be Structurally Deficient or Functionally Obsolete. A sufficiency number greater than 80 is also required after completion of the rehabilitation work. However, if conditions exist that would prevent the completed improvement from correcting all deficiencies, WisDOT shall determine if the proposed project is eligible based on safety and the public interest. Contact the Bureau of Structures Development Section for a waiver of the sufficiency number requirement.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic. Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and riding surface. Wisconsin has experienced some cracking in the concrete overlays and parapets during rehabilitation construction. The apparent cause of cracking is related to traffic impact due to rough structure approaches. Particular attention should be given to providing a smooth transition on temporary approaches and over expansion joints on all bridge decks during rehabilitation. Other causes of cracking are related to shrinkage, temperature, wind induced vibrations, concrete quality and improper curing.

Options for deck rehabilitation are as follows:

1. Asphalt Patch
2. Asphalt or Polymer Modified Asphaltic Overlay
3. Concrete or Modified Concrete Patch
4. Waterproof Membrane with an Asphalt Overlay (currently not used)
5. Concrete Overlay - Grade E Low Slump Concrete, or Micro-Silica Modified Concrete

Consider the following criteria for rehabilitation of highway bridges:

1. Interstate Bridges as Stand Alone Project
    a. Deck condition equal 4 or 5 and;
    b. Wear course or wear surface less than or equal to 3.
    c. No roadway work scheduled for at least 3 years.
2. Interstate Bridge with Roadway Work
    a. No previous work in last 10 years or;
    b. Deck Condition less than or equal 4.
    c. Wear course or wear surface less than or equal to 4.
3. Rehab not needed on Interstate Bridges if:
    a. Deck rehab work less than 10 years old.
    b. Deck condition greater than 4.
    c. Wear surface or wear course greater than or equal 4.

4. All Bridges

WisDOT policy item:

On major rehab work, build to current standards such as safety parapets, full shoulder widths, etc. Use the current Bridge Manual standards and tables. Exceptions to this policy require approval from the Bureau of Structures Development Section.

a. Evaluate cost of repeated maintenance, traffic control as well as bridge work when determining life-cycle costs.

b. Place overlays on all concrete superstructure bridges if eligible.

c. For all deck replacement work the railing shall be built to current standards.

5. All Bridges with Roadway Work

Coordinate with the Region the required staging of bridge related work.

A number of specific guidelines are defined in subsequent sections. As with any engineering project, the engineer is allowed to use discretion in determining the applicability of these guidelines.

40.5 Deck Overlays

If the bridge is a candidate for replacement or a new deck, serviceability may be extended 3 to 7 years by patching and/or overlaying the deck with only a 1-1/2" minimum thickness asphaltic mat on lightly traveled roadways. Experience indicates the asphalt tends to slow down the rate of deterioration while providing a smooth riding surface. However, these decks must be watched closely for shear or punching shear failures as the deck surface problems are concealed.

For applications where the deck is structurally sound and service life is to be extended there are other methods to use. A polymer modified asphaltic overlay may be used to increase deck service life by approximately 15 years. If the concrete deck remains structurally sound, it may be practical to remove the existing overlay and place a new overlay before replacing the deck.

A 1-1/2" concrete overlay is expected to extend the service life of a bridge deck for 15 to 20 years. On delaminated but structurally sound decks a concrete overlay is often the only alternative to deck replacement. Prior to placing the concrete overlay, a minimum of 1" of existing deck surface should be removed. On all bridges low slump Grade E concrete is the specified standard with close inspection of concrete consolidation and curing. If the concrete deck remains structurally sound; it may be practical to remove the existing overlay and place a second deck overlay before replacing the entire deck. After the concrete overlay is placed, it is very important to seal all the deck cracks. Experience shows that salt water passes thru these cracks and causes deterioration of the underlying deck.

On deck overlays preparation of the deck is an important issue after removal of the top surface. Check the latest Special Provisions and/or specifications for the method of payment for Deck Preparation where there are asphalt patches or unsound concrete.

Micro-silica concretes have been effectively used as an alternate type of concrete overlay. It provides excellent resistance to chloride penetration due to its low permeability. Micro-silica modified concrete overlays appear very promising; however, they are still under experimental evaluation. Latex overlays when used in Wisconsin have higher costs without noticeable improved performance.

Ready mixed Grade E concrete with superplasticizer and fiber mesh have been tried and do not perform any better than site mixed concrete produced in a truck mounted mobile mixer.

Bridges with Inventory Ratings less than HS10 with an overlay shall not be considered for concrete overlays, unless approved by the Bureau of Structures Design Section. Bridges reconstructed with overlays shall have their new Iinventory and Operating Ratings shown on the bridge rehabilitation plans. Verify the desired transverse cross slope with the Regions as they may want to use current standards.

40.5.1 Guidelines for Bridge Deck Overlays

As a structure ages, rehabilitation is a necessary part of insuring a level of acceptable serviceability. Overlays can be used to extend the service lives of bridge decks that have surface deficiencies. Guidelines for determining if an overlay should be used are:

- The structure is capable of carrying the overlay deadload;
- The deck and superstructure are structurally sound;
- The desired service life can be achieved with the considered overlay and existing structure;
- The selected option is cost effective based on the structure life.

40.5.2 Deck Overlay Methods

An AC Overlay or Polymer Modified Asphaltic Overlay should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic. All full depth repairs shall be made with PC concrete.

Guidelines for determining the type of deck overlay method to achieve the desired extended service life are:

AC Overlay (ACO): 5 years average life expectancy

- The minimum asphaltic overlay thickness is 1-1/2”.
- The grade change due to overlay thickness can be accommodated at minimal cost.
- Deck or bridge replacement is programmed within 7 years.
- Raising of floor drains or joints is not required.
- Spalls can be patched with AC or PC concrete with minimal surface preparation.

Polymer Modified Asphaltic Overlay: 15 to 20 years life expectancy

- This product may be used as an experimental alternate to LSCO given below. CAUTION – Core tests have shown the permeability of this product is dependent on the aggregate. Limestone should not be used.

Polymer Overlay: 10 to 15 years life expectancy

- A 3/8-inch thick, two layer system comprised of a two-component polymer in conjunction with natural or synthetic aggregates. Use 5 psf for dead load, DW.
- Works well to seal decks and/or provide traction.

The minimum required concrete age is 28 days prior to application, although a longer period of time would allow more initial concrete cracking to occur which the resin would then be able to seal.

AC Overlay with a Waterproofing Membrane (ACOWM): (Currently not used)

Low Slump Concrete Overlay *(LSCO): 15 to 20 years life expectancy

- Minimum thickness is 1-1/2" PC concrete overlay.
- Joints and floor drains will be modified to accommodate the overlay.
- Deck deficiencies will be corrected with PC concrete.
- The prepared deck surfaces will be scarified or shot blasted.
- There is no structural concern for excessive leaching at working cracks.
- Combined distress area is less than 25%.
- May require crack sealing the following year and periodically thereafter.

* Note: Or another PC concrete product as approved by the Bureau of Structures Development Section and coordinated with the Region.

40.5.3 Maintenance Notes

- All concrete overlays crack immediately. If the cracks in the deck are not sealed periodically, the rate of deterioration can increase rapidly.

40.5.4 Special Considerations

On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans.

If more than 1/3 of the steel is exposed, either the centers of adjacent spans must be shored or only longitudinally overlay 1/3 of the bridge at a time.

40.5.5 Railings and Parapets

Overlays increase vehicle lean over sloped face parapets resulting in vehicles on bridges with higher ADT and/or speed having an increased likelihood of impact with lights/obstructions on top of, or behind, the parapet. See Chapter 30-Railings for guidance pertaining to railings and parapets associated with rehabilitation structures projects.

Sub-standard railings and parapets should be improved. An example of such a sub-standard barrier would be a curb with a railing or parapet on top. Contact the Bureau of Structures Development Section to discuss solutions.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. Refer to the criteria in Section 40.3 of this chapter for additional 3R project considerations. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:

Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating greater than HS18, but less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the *Facilities Development Manual* and *FDM SDD 14b7* for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, only use intermediate steel diaphragms in locations of removed intermediate concrete diaphragms (i.e. don’t add intermediate lines of diaphragms).

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 45”, 54” and 70” girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections’ draped and undraped strand patterns.

The 45”, 54”, and 70” girders in Chapter 40-Bridge Rehabilitation standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at $0.75f_{pu}$,
- A concrete haunch of 2-1/2",
- Slab thicknesses from Chapter 17-Superstructure - General,
- A future wearing surface of 20 psf,
- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads,
- 0.5” or 0.6” dia. strands (in accordance with the Standard Details),
- f'_c girder = 8,000 psi,
- f'_c slab = 4,000 psi, and
- Required f'_c girder at initial prestress < 6,800 psi

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	102	112
6’-6”	100	110
7’-0”	98	108
7’-6”	96	102
8’-0”	94	100
8’-6”	88	98
9’-0”	88	96
9’-6”	84	90
10’-0”	84	88
10’-6”	82	86
11’-0”	78	85
11’-6”	76	84
12’-0”	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	130	138
6’-6”	128	134
7’-0”	124	132
7’-6”	122	130
8’-0”	120	128
8’-6”	116	124
9’-0”	112	122
9’-6”	110	118
10’-0”	108	116
10’-6”	106	112
11’-0”	102	110
11’-6”	100	108
12’-0”	98	104

70” Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	150*	160*
6’-6”	146*	156*
7’-0”	144*	152*
7’-6”	140*	150*
8’-0”	138*	146*
8’-6”	134*	142*
9’-0”	132*	140*
9’-6”	128*	136
10’-0”	126*	134
10’-6”	122	132
11’-0”	118	128
11’-6”	116	126
12’-0”	114	122

Table 40.7-1
Maximum Span Length vs. Girder Spacing

*For lateral stability during lifting these girder lengths will require pick up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.

40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. Also, reference is made to the criteria in Section 40.3 of this Chapter for additional 3R project considerations. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, the total deck should be replaced in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the life-cycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W" rather than 54"). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. *The girders used for widenings may be the latest Chapter 19-Prestressed Concrete sections designed to LRFD or the sections from Chapter 40-Bridge Rehabilitation designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.*

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet AASHTO 3.6.5 (600 kip loading) as a widening is considered rehabilitation. *BOS intends to provide standard details in the Bridge Manual for a crash barrier that could, at the option of the Region, be used to strengthen and provide motorists protection for existing piers, including widenings.*

Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development Section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don't add intermediate lines of diaphragms).

40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3' or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab. Approval is required from BOS for all Superstructure replacement projects. Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading).

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 600 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 600 kip vehicular impact loading need not be considered if a minimum of "vehicle protection" is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

For abutments, evaluate the piles, or bearing capacity of the ground if on spread footings, utilizing Service I loading. The abutment body should be evaluated using Strength I loading.

The superstructure shall be designed to current LRFD criteria.

40.10 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.

40.11 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 600 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 600 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 600 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 600 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

Timber abutments consist of a single row of piling capped with timber or concrete, and backed with timber to retain the approach fill. The superstructure types are generally concrete slab or timber. Timber-backed abutments currently exist on Town Roads and County Highways where the abutment height does not preclude the use of timber backing.

Piles in bents are designed for combined axial load and bending moments. For analysis, the assumption is made that the piles are supported at their tops and are fixed 6 feet below the stream bed or original ground line. For cast-in-place concrete piling, the concrete core is designed to resist the axial load. The bending stress is resisted by the steel shell section. Due to the possibility of shell corrosion, steel reinforcement is placed in the concrete core equivalent to a 1/16-inch steel shell perimeter section loss. The reinforcement design is based on equal section moduli for the two conditions. Reinforcement details and bearing capacities are given on the Standard Detail for Pile Details. Pile spacing is generally limited to the practical span lengths for timber backing planks.

The requirements for tie rods and deadmen is a function of the abutment height. Tie rods with deadmen on body piling are used when the height of "freestanding" piles is greater than 12 feet for timber piling and greater than 15 feet for cast-in-place concrete and steel "HP" piling. The "freestanding" length of a pile is measured from the stream bed or berm to grade. If possible, all deadmen should be placed against undisturbed soil.

Commercial grade lumber as specified in AASHTO having a minimum flexural resistance of 1.2 ksi is utilized for the timber backing planks. The minimum recommended nominal thickness and width of timber backing planks are 3 and 10 inches, respectively. If nominal sizes are specified on the plans, analysis computations must be based on the dressed or finished sizes of the timber. Design computations can be used on the full nominal sizes if so stated on the bridge plans. For abutments constructed with cast-in-place concrete or steel "HP" piles, the timber planking is attached with 60d common nails to timber nailing strips which are bolted to the piling.

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

Existing steel expansion devices shall be modified or replaced with Watertight Expansion Devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6' or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide down hill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

40.14 Superstructure Inspection

40.14.1 Prestressed Girders

On occasion prestressed concrete girders are damaged during transport, placement, or as a result of vehicle impact. Damage inspection and assessment procedures are necessary to determine the need for traffic restrictions, temporary falsework for safety and/or strength. Three predominant assessment areas are damage to prestressing strands, damage to concrete, and remaining structural integrity.

Where damage to a girder results in any significant loss of concrete section, an engineering analysis should be made. This analysis should include stress calculations for the damaged girder with comparison to the original design stresses. These calculations will show the loss of strength from the damage.

Assessment of damage based on the loss of one or more prestressing strands or loss of prestress force is given as reason for restriction or replacement of girders. Some of the more common damages are as follows with the recommended maintenance action. However, an engineering assessment should be made on all cases.

1. If cracking and spalling are limited to the lower flange, patching is usually performed. This assessment should be based on calculations that may allow repair.

2. If cracking continues from flange into web, the girder is normally replaced. Findings indicate that sometimes repair-in-place may be the preferred decision.

3. Termini of cracks are marked, and if the cracks continue to grow the girder may be replaced. This is a good method to determine the effect of loads actually being carried.

4. When large areas of concrete are affected, or when concrete within stirrups is fractured, replace. At times it may be more appropriate to repair-in-place than replace.

Repair and/or replacement decisions are based on structural integrity. Load capacity is by far the most important rationale for selection of repair methods. Service load capacity needs to be calculated if repair-in-place is contemplated.

Evaluations of criteria for assessment of damage related to the two predominant areas associated with structural integrity are the following:

1. A structural analysis is made to determine the stresses in a damaged girder on the basis of the actual loads the girder will carry. This technique may result in a girder that has less capacity than the original, but can still safely carry the actual loads. This assessment results in consideration of possible repair-in-place methods which normally are cost effective.

   or

2. A structural analysis is made to determine the load capacity and rating of the girder. If the capacity and rating of the girder is less than provided by the original design, the girder shall be replaced. This assessment will provide a girder equal to the original design, but precludes possible repair-in-place methods that are normally less costly.

Location and size of all spalled and unsound concrete areas shall be recorded. Location, length, and width of all visible cracks shall be documented. All damage to prestress strands and reinforcing steel shall be reported. Location of hold-down devices in the girder shall be shown in relation to the damage. Horizontal and vertical misalignment along the length of the girder, and at points of localized damage, shall be reported. (These measurements might best be made by string-lining). Growth of cracks shall be monitored to determine that the cracked section has closed before extending to the web.

Critical damage is damage to concrete and/or the reinforcing elements of prestressed concrete girders such as:

1. Cracks extend across the bottom flange and/or in the web directly above the bottom flange. (This indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset is measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.

4. Vertical misalignment in excess of the normal allowable.

5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface. (This indicates permanent deformation of stirrups).

40.14.2 Steel Beams

These are three alternate methods of repairing damaged steel beams. They are:

1. Replace the total beam,

2. Replace a section of the beam, or

3. Straighten the beam in-place by heating and jacking.

The first alternate would involve removing the concrete deck over the damaged beam, remove the damaged section and weld in a new piece; then reconstruct the deck slab and railing over the new girder. Falsework support is required at the locations where the beams are cut and probably in the adjacent span due to an unbalanced condition.

The second alternate involves cutting out a section of the beam after placing the necessary supporting members. The support is placed using calibrated jacks. The section is cut out as determined by the damage. A new section plus any vertical stiffeners and section of cover plates would be welded in. This involves butt welds on both the flange and web. The welding of the web is difficult due to minor misalignments to start with plus the tendency of thin plates to move from the heat of welding.

The third alternate of heating and jacking the in-place beam to straighten it is a difficult procedure but can be done by personnel familiar and knowledgeable of the process. It is important to maintain heat control under 1300°F maximum. Use an optical pyrometer to determine heat temperature. There is no specified tolerance for the straightened member. The process is deemed satisfactory when a reasonable alignment is obtained.

Based on the three alternates available, the estimated costs involved and the resultant restoration of the beam to perform its load carrying capacity, heat straightening is a viable option in many cases.

The structural engineer who will be responsible for plan preparation should field review the site with the Regional Bridge Maintenance Engineer.

40.15 Substructure Inspection

The inspection of substructure components may reveal deteriorating concrete in areas exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows. Footings and pilings exposed due to erosion and undermining could result in loss of bearing capacity and/or section.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Reuse of steel pile sections will require checking the remaining allowable load carrying capacity. Steel piling should be checked immediately below the splash zone or water line for deterioration and possible loss of section. High section loss has occurred in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line. Bearing capacities of existing footings and pilings may have to be recomputed in order to determine if superstructure loading can be safely carried.

Timber substructure components may exhibit deterioration due to fungus decay, abrasion wear and weathering. Also, physical damage may be caused by vermin attacks, chemicals, fires, and collisions. Prior to reuse, timber backed abutments and pier bents shall be checked, by boring, for material and mechanical condition, section loss and structural adequacy.

40.15.1 Hammerhead Pier Rehabilitation

Pier caps and sometimes shafts of these piers have become spalled due to leaky joints in the deck. The spalling may be completely around some of the longitudinal bar steel destroying the bond. However, experience shows that the concrete usually remains sound under the bearing plates. There is no known reason for this except that maybe the compressive forces may prevent salt intrusion or counteract freeze thaw cycles.

If the longitudinal bars are full length, the bond in the ends insures integrity even though spalling may occur over the shaft. Corrective action is required as follows:

1. Place a watertight expansion joint in the deck.
2. Consider whether bearing replacement is required.
3. Analyze the type of cap repair required.
    a. Clean off spalled concrete and place new concrete.
    b. Analyze capacity of bars still bonded to see if unbonded bars are needed. Use ultimate strength analysis.
    c. Consider repair method for serious loss of bar steel capacity.
        i. Add 6" of cover to cap. Add additional bar steel. Grout in U shaped stirrups around bars using standard anchor techniques.

ii. Use steel plates and post-tensioning bars to place compression loads on both ends of cap. Cover exposed bars with concrete.

iii. Pour wing extension under pier caps beginning at base to take all loads in compression. This would alter pier shape.

d. Consider sloping top of pier to get better drainage.

e. Consider placing coating on pier top to resist water intrusion.

If the shaft is severely spalled, it will require a layer of concrete to be placed around it. Otherwise, patching is adequate. To add a layer of concrete:

1. Place anchor bars in existing solid concrete. Use expansion anchors or epoxy anchors as available.

2. Place wire mesh around shaft.

3. Place forms and pour concrete. 6” is minimum thickness.

40.15.2 Bearings

All steel bridge bearings should be replaced as shown in Chapter 27-Bearings. Compressive load and adhesion tests will be waived for steel laminated elastomeric bearings where these bearings are detailed to meet height requirements.

In general replace lubricated bronze bearings with Teflon bearings. If only outside bearings are replaced, the difference in friction factors can be ignored. Where lubricated bronze bearings might be used, following is the design criteria.

For the expansion bearings, two additional plates are employed over fixed bearings, a top plate and a lubricated bronze plate. Current experience indicates that a stainless steel top plate reduces corrosion activity and is the recommended alternate to steel. The top plate is set on top of a lubricated bronze plate allowing expansion and contraction to occur. Laboratory testing of lubricated bronze plates indicated a maximum coefficient of friction varying from 8 to 14 percent for a loading of 200 kips. Current BOS practice for steel girder Type “A” and prestressed girder expansion bearings of employing a 10 percent maximum and a 6 percent minimum friction value for design is in accordance to laboratory test results. For Type “A” bearing details refer to Standard Details.

40.16 Concrete Masonry Anchors for Rehabilitation

"Type S" and "Type L" concrete masonry anchors are used mostly for bridge rehabilitation projects and anchoring pedestrian rail posts. The main difference between the two types of anchors is the duration of loading. The "Type S" anchors are used for Short-term loading conditions and the "Type L" anchors are used for Long-term loading.

The minimum pullout capacity (Nominal Tensile Resistance) of a single concrete masonry anchor is determined according to 40.16.4; however, this value is only specified on the plan for "Type S" mechanical anchors. The minimum pullout capacity is not specified on the plan for "Type S" or "Type L" adhesive anchors because the anchors must be designed to meet the minimum bond stresses as noted in Table 40.16-1. If additional capacity is required, a more refined analysis (i.e., anchor group analysis) per the current version of ACI 318 Appendix D is allowable, which may yield higher capacities. References in this section, **ACI [D._]** refer to articles within the ACI 318 manual. (*AASHTO* currently does not have guidance for anchors.)

40.16.1 Adhesive Anchor Requirements

For all non-mechanical anchors, there are two processes used to install the adhesive. One option uses a two-part adhesive that is mixed and poured into the drilled hole. The second option pumps a two-part adhesive into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. With either process, the hole must be properly cleaned and a sufficient amount of adhesive used so that the hole is completely filled with adhesive when the rebar or bolt is inserted. The adhesive bond stresses, as noted in Table 40.16-1, are determined by the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 or ACI 355.4.

40.16.2 Concrete Masonry Anchor Types and Usage

"Type S" anchors are either mechanical wedge or adhesive anchors for installing studs, rebar, or bolts of a designated size. They are typically smaller size rebars and are primarily used for anchoring bolts for attaching rail posts or other bolted objects. "Type S" mechanical wedge anchors are seldom used for bridge rehabilitation. When "Type S" anchors are used to anchor rebars, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

"Type L" anchors are adhesive anchors used to anchor rebars when the rebar is subject to continuous loading. "Type L" anchors are typically used for abutment and pier widenings, but may be used in other applications. Because of creep, shrinkage, and deterioration under load and freeze-thaw cycles, the Approved Products List for "Type L" anchors has been assembled to account for rebar experiencing a constant tension stress. For "Type L" anchors the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

Per **ACI [D.1]**, concrete masonry anchor steel is considered ductile if the tensile test elongation is at least 14 percent and reduction in area is at least 30 percent. Additionally, steel meeting the requirements of ASTM A307 is considered ductile. Steel that does not meet these requirements is considered brittle. Rebar used as anchor steel is considered ductile.

Usage Restrictions: Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers. **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column) or for vertical overhead installations.**

The manufacturer and product name of the “Type L” anchors and “Type S” adhesive anchors used by the contractor must be on the Department’s approved product list for “Concrete Masonry Anchors, Type L”.

The required minimum anchor spacing is 6 times the diameter of the anchor. The minimum edge distance is 6 times the diameter of the anchor for adhesive anchors and 10 times the diameter for mechanical anchors. The minimum member thickness for mechanical anchors is the greater of the embedment depth plus 4 inches and 3/2 of the embedment depth. The maximum embedment depth for adhesive anchors is 20 times the anchor diameter.

40.16.3 Concrete Masonry Anchor Reinforcement

Reinforcement used to transfer the full design load from the anchors into the structural member is considered anchor reinforcement. **ACI [D.5.2.9]** and **ACI [D.6.2.9]** provide guidance for designing anchor reinforcement. When anchor reinforcement is used, the concrete breakout strength need not be checked per 40.16.4 and 40.16.5. Reinforcement that acts to restrain the potential concrete breakout but is not designed to transfer the full design load is considered to be supplementary reinforcement.

40.16.4 Concrete Masonry Anchor Tensile Capacity

Concrete masonry anchors in tension fail in one of four ways: steel tensile rupture, concrete breakout, pullout strength of anchors in tension, or adhesive bond. The pullout strength of anchors in tension only applies to mechanical anchors and the adhesive bond only applies to adhesive anchors. Figure 40.16-1 shows the concrete breakout failure mechanism for anchors in tension.

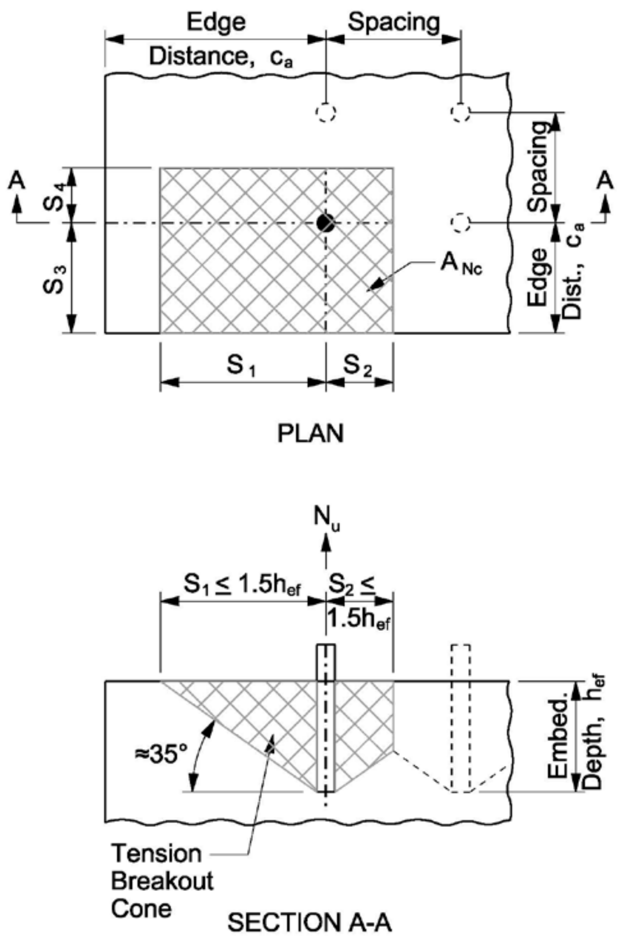

Figure 40.16-1

Concrete Breakout of Concrete Masonry Anchors in Tension

The projected concrete breakout area, A_{Nc}, shown in Figure 40.16-1 is limited in each direction by S_i:

S_i = Minimum of:

1. 1.5 times the embedment depth (h_{ef}),
2. Half of the spacing to the next anchor in tension, or
3. The edge distance (c_a) (in).

Figure 40.16-2 shows the bond failure mechanism for concrete masonry adhesive anchors in tension.

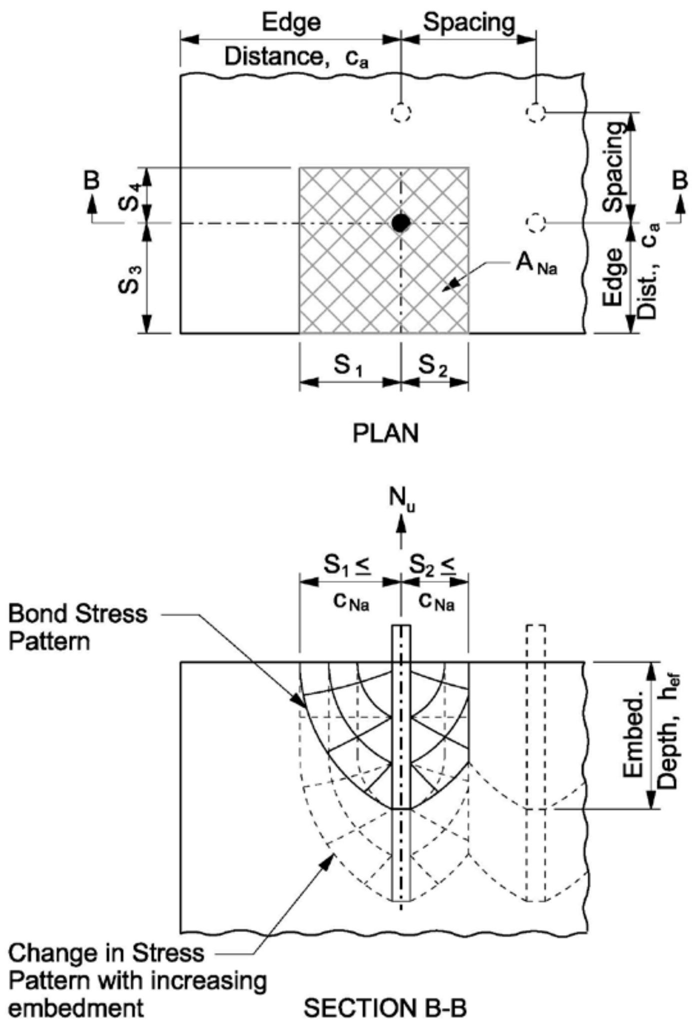

Figure 40.16-2

Bond Failure of Concrete Masonry Adhesive Anchors in Tension

The projected influence area of a single adhesive anchor, A_{Na}, is shown in Figure 40.16-2. Unlike the concrete breakout area, it is not affected by the embedment depth of the anchor. A_{Na} is limited in each direction by S_i:

S_i = Minimum of:

1. $c_{Na} = 10 d_a \sqrt{\dfrac{\tau_{uncr}}{1100}}$,

2. Half of the spacing to the next anchor in tension, or

3. The edge distance (c_a) (in).

	"Type S" Anchors	"Type S" or "Type L" Anchors			
	Mechanical	Adhesive (Dry Concrete)		Adhesive (Water-Saturated Concrete)	
Anchor Size, d_a	Min. h_{ef} (in)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)	Min. Bond Stress, τ_{uncr} (psi)	Min. Bond Stress, τ_{cr} (psi)
#4 or 1/2"	2.5	1290	871	510	386
#5 or 5/8"	2.5	1290	907	722	522
#6 or 3/4"	3.0	1290	907	709	506
#7 or 7/8"	3.5	1280	815	682	434
#8 or 1"	4.0	1215	800	807	531
#9 or 1-1/8"	4.5	1025	700	681	465

Table 40.16-1

Tension Design Table for Concrete Masonry Anchors, "Type S" and "Type L"

The minimum bond stress values for adhesive anchors in Table 40.16-1 are based on the Approved Products List for "Concrete Masonry Anchors, Type L". The designer shall determine whether the concrete masonry adhesive anchors are to be utilized in dry concrete (i.e., rehabilitation locations where concrete is fully cured, etc.) or water-saturated concrete (i.e., new bridge decks, box culverts, etc.) and shall design the anchors accordingly.

The factored tension force on each anchor, N_u, must be less than or equal to the factored tensile resistance, N_r. For mechanical anchors:

$$N_r = \phi_{ts} N_{sa} \le \phi_{tc} N_{cb} \le \phi_{tc} N_{pn}$$

In which:

ϕ_{ts} = Strength reduction factor for anchors in concrete, **ACI [D.4.3]**
= 0.65 for brittle steel as defined in 40.16.2
= 0.75 for ductile steel as defined in 40.16.2

N_{sa} = Nominal steel strength of anchor in tension, **ACI [D.5.1.2]**
= $A_{se,N} f_{uta}$

$A_{se,N}$ = Effective cross-sectional area of anchor in tension (in^2)

f_{uta} = Specified tensile strength of anchor steel (psi)
≤ $1.9f_{ya}$
≤ 125 ksi

f_{ya} = Specified yield strength of anchor steel (psi)

ϕ_{tc} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.3
= 0.75 for anchors with supplementary reinforcement per 40.16.3

N_{cb} = Nominal concrete breakout strength in tension, **ACI [D.5.2.1]**
= $\frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

A_{Nc} = Projected concrete failure area of a single anchor, see Figure 40.16-1
= $(S_1+S_2)(S_3+S_4)$

h_{ef} = Effective embedment depth of anchor per Table 40.16-1. May be reduced per **ACI [D.5.2.3]** when anchor is located near three or more edges.

$\psi_{ed,N}$ = Modification factor for tensile strength based on proximity to edges of concrete member, **ACI [D.5.2.5]**
= 1.0 if $c_{a,min} \geq 1.5h_{ef}$
= $0.7+0.3\frac{c_{a,min}}{1.5h_{ef}}$ if $c_{a,min} < 1.5h_{ef}$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 (in)

$\psi_{c,N}$ = Modification factor for tensile strength of anchors based on the presence or absence of cracks in concrete, **ACI [D.5.2.6]**
= 1.0 when post-installed anchors are located in a region of a concrete member where analysis indicates cracking at service load levels
= 1.4 when post-installed anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [D.5.2.7]**
= 1.0 if $c_{a,min} \geq c_{ac}$
= $\frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

c_{ac} = Critical edge distance (in)
= $4.0h_{ef}$

N_b = Concrete breakout strength of a single anchor in tension in uncracked concrete, **ACI [D.5.2.2]**
= $0.538\sqrt{f'_c}(h_{ef})^{1.5}$ (kips)

N_{pn} = Nominal pullout strength of a single anchor in tension, **ACI [D.5.3.1]**
= $\psi_{c,P}N_p$

$\psi_{c,P}$ = Modification factor for pullout strength of anchors based on the presence or absence of cracks in concrete, **ACI [D.5.3.6]**
= 1.4 where analysis indicates no cracking at service load levels
= 1.0 where analysis indicates cracking at service load levels

N_p = Nominal pullout strength of a single anchor in tension based on the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC193 / ACI 355.2

For adhesive anchors:

$$N_r = \phi_{ts}N_{sa} \le \phi_{tc}N_{cb} \le \phi_{tc}N_a$$

In which:

N_{cb} = Nominal concrete breakout strength in tension, **ACI [D.5.2.1]**
= $\frac{A_{Nc}}{9(h_{ef})^2}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

h_{ef} = Effective embedment depth of anchor. May be reduced per **ACI [D.5.2.3]** when anchor is located near three or more edges.
≤ $20d_a$ (in)

d_a = Outside diameter of anchor (in)

$\psi_{cp,N}$ = Modification factor for post-installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [D.5.2.7]**
= 1.0 if $c_{a,min} \ge c_{ac}$
= $\frac{c_{a,min}}{c_{ac}} \ge \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

$c_{a,min}$ = Minimum edge distance from center of anchor shaft to the edge of concrete, see Figure 40.16-1 or Figure 40.16-2 (in)

c_{ac} = Critical edge distance (in)
= $2.0h_{ef}$

N_a = Nominal bond strength of a single anchor in tension, **ACI [D.5.5.1]**
= $\frac{A_{Na}}{4c_{Na}^2}\psi_{ed,Na}\psi_{cp,Na}N_{ba}$

A_{Na} = Projected influence area of a single adhesive anchor, see Figure 40.16-2
= $(S_1 + S_2)(S_3 + S_4)$

$\psi_{ed,Na}$ = Modification factor for tensile strength of adhesive anchors based on the proximity to edges of concrete member, **ACI [D.5.5.4]**
= 1.0 if $c_{a,min} \geq c_{Na}$
= $0.7 + 0.3\frac{c_{a,min}}{c_{Na}}$ if $c_{a,min} < c_{Na}$

c_{Na} = Projected distance from center of anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor
= $10d_a\sqrt{\frac{\tau_{uncr}}{1100}}$ (in)

τ_{uncr} = Characteristic bond stress of adhesive anchor in uncracked concrete, see Table 40.16-1

$\psi_{cp,Na}$ = Modification factor for pullout strength of adhesive anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation, **ACI [D.5.5.5]**
= 1.0 if $c_{a,min} \geq c_{ac}$
= $\frac{c_{a,min}}{c_{ac}} \geq \frac{c_{Na}}{c_{ac}}$ if $c_{a,min} < c_{ac}$

N_{ba} = Bond strength in tension of a single adhesive anchor, **ACI [D.5.5.2]**
= $\tau_{cr}\pi d_a h_{ef}$

τ_{cr} = Characteristic bond stress of adhesive anchor in cracked concrete based on the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 / ACI 355.4, see Table 40.16-1

Note: Where analysis indicates cracking at service load levels, adhesive anchors shall be qualified for use in cracked concrete in accordance with ICC-ES AC308 / ACI

355.4. For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr} and shall be taken as the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 / ACI 355.4.

In addition to the checks listed above for all adhesive anchors, for “Type L” anchors, the factored sustained tensile force must be less than or equal to the factored sustained tensile resistance per **ACI [D.4.1.2]**:

$$0.55\phi_{tc}N_{ba} \geq N_{ua,s}$$

40.16.5 Concrete Masonry Anchor Shear Capacity

Concrete masonry anchors in shear fail in one of three ways: steel shear rupture, concrete breakout, or concrete pryout. Figure 40.16-3 shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in Figure 40.16-3 is limited vertically by H, and in both horizontal directions by S_i:

H = Minimum of:

1. The member depth (h_a) or
2. 1.5 times the edge distance (c_{a1}) (in).

S_i = Minimum of:

1. Half the anchor spacing (S),
2. The perpendicular edge distance (c_{a2}), or
3. 1.5 times the edge distance (c_{a1}) (in).

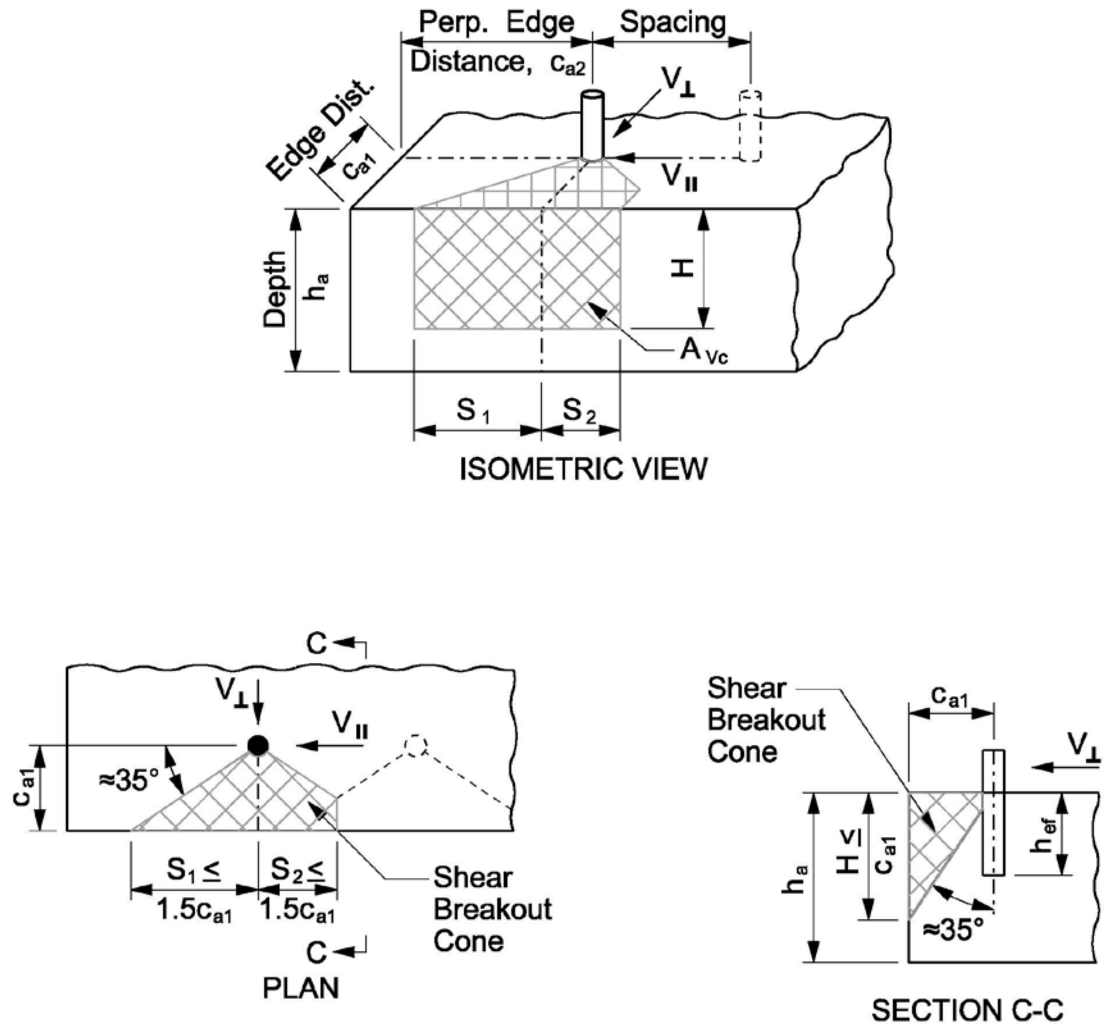

Figure 40.16-3

Concrete Breakout of Concrete Masonry Anchors in Shear

If the shear is applied to more than one row of anchors as shown in Figure 40.16-4, the shear capacity must be checked for the worst of the three cases. If the row spacing, SP, is at least equal to the distance from the concrete edge to the front anchor, E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the back anchor is checked for the full shear load. If the row spacing, SP, is less than the distance from the concrete edge to the front anchor, E1, then check Case 3. In case 3, the front anchor is checked for the full shear load. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.

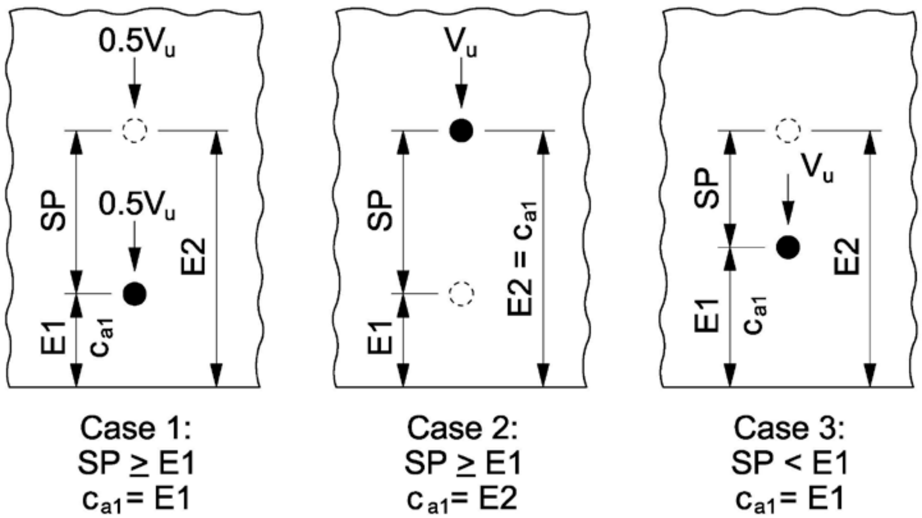

Figure 40.16-4
Concrete Masonry Anchor Shear Force Cases

The factored shear force on each anchor, V_u, must be less than or equal to the factored shear resistance, V_r. For mechanical and adhesive anchors:

$$V_r = \phi_{vs} 0.6 V_{sa} \leq \phi_{vc} V_{cb} \leq \phi_{vp} V_{cp}$$

In which:

ϕ_{vs} = Strength reduction factor for anchors in concrete, **ACI [D.4.3]**
= 0.60 for brittle steel as defined in 40.16.2
= 0.65 for ductile steel as defined in 40.16.2

V_{sa} = Nominal steel strength of anchor in shear, **ACI [D.6.1.2]**
= $A_{se,V} f_{uta}$

$A_{se,V}$ = Effective cross-sectional area of anchor in tension (in^2)

ϕ_{vc} = Strength reduction factor for anchors in concrete
= 0.70 for anchors without supplementary reinforcement per 40.16.3
= 0.75 for anchors with supplementary reinforcement per 40.16.3

V_{cb} = Nominal concrete breakout strength in shear, **ACI [D.6.2.1]**
= $\dfrac{A_{Vc}}{4.5(c_{a1})^2} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{p,V} V_b$

A_{Vc} = Projected area of the concrete failure surface on the side of the concrete member at its edge for a single anchor, see Figure 40.16-3
= $H(S_1 + S_2)$

c_{a1} = Distance from the center of anchor shaft to the edge of concrete in the direction of the applied shear, see Figure 40.16-3 and Figure 40.16-4 (in)

$\psi_{ed,V}$ = Modification factor for shear strength of anchors based on proximity to edges of concrete member, **ACI [D.6.2.6]**
= 1.0 if $c_{a2} \geq 1.5c_{a1}$ (perpendicular shear)
= $0.7 + 0.3\dfrac{c_{a2}}{1.5c_{a1}}$ if $c_{a2} < 1.5c_{a1}$ (perpendicular shear)
= 1.0 (parallel shear)

c_{a2} = Distance from the center of anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, see Figure 40.16-3 (in)

$\psi_{c,V}$ = Modification factor for shear strength of anchors based on the presence or absence of cracks in concrete and the presence or absence of supplementary reinforcement, **ACI [D.6.2.7]**
= 1.4 for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels
= 1.0 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels without supplementary reinforcement per 40.16.3 or with edge reinforcement smaller than a No. 4 bar
= 1.2 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge
= 1.4 for anchors located in a region of a concrete member where analysis indicates cracking at service load levels with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at no more than 4 inches

$\psi_{h,V}$ = Modification factor for shear strength of anchors located in concrete members with $h_a < 1.5_{ca1}$, **ACI [D.6.2.8]**
= $\sqrt{\dfrac{1.5c_{a1}}{h_a}} \geq 1.0$

h_a = Concrete member thickness in which anchor is located measured parallel to anchor axis, see Figure 40.16-3 (in)

$\psi_{p,V}$ = Modification factor for shear strength of anchors based on loading direction, **ACI [D.6.2.1(c)]**
= 1.0 for shear perpendicular to the concrete edge, see Figure 40.16-3
= 2.0 for shear parallel to the concrete edge, see Figure 40.16-3

V_b = Concrete breakout strength of a single anchor in shear in cracked concrete, per **ACI [D.6.2.2]**, shall be the smaller of:

$$[7(\frac{l_e}{d_a})^{0.2}\sqrt{d_a}]\sqrt{f'_c}(c_{a1})^{1.5} \text{ (lb)}$$

Where:
$l_e = h_{ef} \leq 8d_a$
d_a = Outside diameter of anchor (in)
f'_c = Specified compressive strength of concrete (psi)

and

$$9\sqrt{f'_c}(c_{a1})^{1.5}$$

ϕ_{vp} = Strength reduction factor for anchors in concrete
= 0.65 for anchors without supplementary reinforcement per 40.16.3
= 0.75 for anchors with supplementary reinforcement per 40.16.3

V_{cp} = Nominal concrete pryout strength of a single anchor, **ACI [D.6.3.1]**
= $2.0N_{cp}$

Note: The equation above is based on $h_{ef} \geq 2.5$ in. All concrete masonry anchors must meet this requirement.

N_{cp} = Nominal concrete pryout strength of an anchor taken as the lesser of:

mechanical anchors: $\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$

adhesive anchors: $\frac{A_{Na}}{4(c_{Na})^2}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$

and

$\frac{A_{Nc}}{9(h_{ef})^2}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$

For shear in two directions, check both the parallel and the perpendicular shear capacity. For shear on an anchor near a corner, check the shear capacity for both edges and use the minimum.

40.16.6 Interaction of Tension and Shear

For both "Type L" and "Type S" anchors that are subjected to tension and shear, interaction equations must be checked per **ACI [D.7]**.

If $\frac{V_{ua}}{\phi V_n} \leq 0.2$ for the governing strength in shear, then the full strength in tension is permitted: $\phi N_n \geq N_{ua}$. If $\frac{N_{ua}}{\phi N_n} \leq 0.2$ for the governing strength in tension, then the full strength in shear is permitted: $\phi V_n \geq V_{ua}$. If $\frac{V_{ua}}{\phi V_n} > 0.2$ for the governing strength in shear and $\frac{N_{ua}}{\phi N_n} > 0.2$ for the governing strength in tension, then:

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$

40.16.7 Plan Preparation

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance as determined in 40.16.4.

Typical notes for bridge plans (shown in all capital letters):

Mechanical "Type S" anchors located in uncracked concrete:

MASONRY ANCHORS TYPE S X/X-INCH. MIN. PULLOUT CAPACITY OF XX KIPS. EMBED XX" IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Mechanical "Type S" anchors located in cracked concrete:

MASONRY ANCHORS TYPE S X/X-INCH. MIN. PULLOUT CAPACITY OF XX KIPS. EMBED XX" IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive "Type S" anchors located in uncracked concrete:

MASONRY ANCHORS TYPE S X/X-INCH. EMBED XX" IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive "Type S" anchors located in cracked concrete:

MASONRY ANCHORS TYPE S X/X-INCH. EMBED XX" IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

When using "Type S" anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item "Masonry Anchors Type S _-Inch".

For "Type S" anchors using rebar, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

Adhesive "Type L" anchors located in uncracked concrete:

MASONRY ANCHORS TYPE L NO. X BARS. EMBED XX" IN CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

Adhesive "Type L" anchors located in cracked concrete:

MASONRY ANCHORS TYPE L NO. X BARS. EMBED XX" IN CONCRETE. ANCHORS SHALL BE APPROVED FOR USE IN CRACKED CONCRETE. *(Illustrative only, values must be calculated depending on the specific situation).*

For "Type L" anchors, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

It should be noted that AASHTO is considering adding specifications pertaining to concrete masonry anchors. This chapter will be updated once that information is available.

40.17 Plan Details

1. Excavation for Structures on Overlays

   There is considerable confusion on when to use or not use the bid item “Excavation for Structures” on overlay projects. In order to remove the confusion, the following note is to be added to all overlay projects that only involve removal of the paving block (or less).

   > Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item “Concrete Masonry Overlay Decks”.

   If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay, the “Excavation for Structures” bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements, show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

   Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by current standard of a 0.02 ft/ft cross slope, a cross slope of 0.01 ft/ft or 0.015 ft/ft may be the most desirable.

   The designer should evaluate 3 types of repairs. “Preparation Decks Type 1” is concrete removal to the top of the bar steel. “Preparation Decks Type 2” is concrete removal below the bar steel. “Full Depth Deck Repair” is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of “Full Depth Deck Repair” on the plan sheet.

   Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction, consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

   For all Bituminous Material Overlays:

Surface Preparation for Sheet Membrane Waterproofing	Area of Deck
Sheet Membrane Waterproofing	Area of Deck
HMA Pavement Type E-xx	Check (E-xx) with Region
Asphaltic Material PGxx-xx	Check (PGxx-xx) with Region
If Asked for in Structure Survey Report	
Preparation Decks Type 1 & 2	If Blank, call Region
Concrete Masonry, Deck Patching	Use 1/2 Slab Thickness
Sawing Pavement, Deck Preparation, Curb or Joint Repair	If asked for by Region

Table 40.17-1
Quantities for Asphaltic Overlays

40.18 Retrofit of Steel Bridges

Out of plane bending stresses may occur in steel bridges that introduce early fatigue cracks or worse yet brittle fractures. Most, if not all, problems are caused at connections that are either too flexible or too rigid. It is important to recognize the correct condition as retrofitting for the wrong condition will probably make it worse.

40.18.1 Flexible Connections

A connection is too flexible when minor movement can occur in a connection that is designed to be rigid. Examples are transverse or bearing stiffeners fitted to a tension flange, floorbeams attached to the web only and not both flanges.

The solution for stiffeners and transverse connection plates is to shop weld the stiffener to the web and flanges. This becomes a Category C weld but is no different than the terminated web to stiffener weld. If the condition exists in the field, welding is not recommended. Retrofit is to attach a T-section with four bolts to the stiffener and four bolts to the flange. Similar details should be used in attaching floorbeams to the girder.

40.18.2 Rigid Connections

A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.

Other partial constraint details are:

1. Intersecting welds
2. Gap size-allowing local yielding
3. Weld size
4. Partial penetration welds versus fillet welds
5. Touching and intersecting welds

The solution is to create spaces large enough (approximately 1/4” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than 1/4” and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

40.19 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

Effective Span Ft-In	T=Slab Thickness Inches	Transverse Bars & Spacing	Longitudinal Bars & Spacing	Longitudinal* Continuity Bars & Spacing
4-0	6.5	#5 @ 8"	#4 @ 8.5"	#5 @ 7.5"
4-3	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-6	6.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 7.5"
4-9	6.5	#5 @ 7"	#4 @ 7.5"	#5 @ 7.5"
5-0	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-3	6.5	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
5-6	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
5-9	6.5	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
2-3	7	#4 @ 9"	#4 @ 11"	#5 @ 6.5"
2-6	7	#4 @ 8.5"	#4 @ 11"	#5 @ 6.5"
2-9	7	#4 @ 8"	#4 @ 11"	#5 @ 6.5"
3-0	7	#4 @ 7.5"	#4 @ 11"	#5 @ 6.5"
3-3	7	#4 @ 7"	#4 @ 11"	#5 @ 6.5"
3-6	7	#4 @ 6.5"	#4 @ 11"	#5 @ 6.5"
3-9	7	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6.5"
4-0	7	#4 @ 6"	#4 @ 10"	#5 @ 6.5"
4-3	7	#5 @ 9"	#4 @ 9.5"	#5 @ 7"
4-6	7	#5 @ 8.5"	#4 @ 9"	#5 @ 7"
4-9	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
5-0	7	#5 @ 8"	#4 @ 8.5"	#5 @ 7"
4-3	7	#5 @ 7.5"	#4 @ 8"	#5 @ 7"
5-6	7	#5 @ 7'	#4 @ 7"	#5 @ 7"
5-9	7	#5 @ 7"	#4 @ 7"	#5 @ 7"
6-0	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-3	7	#5 @ 6.5"	#4 @ 7"	#5 @ 7"
6-6	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
6-9	7	#5 @ 6"	#4 @ 6.5"	#5 @ 6.5"
7-0	7	#5 @ 6"	#4 @ 6"	#5 @ 6"
4-0	7.5	#4 @ 7"	#4 @ 10.5"	#5 @ 6"
4-3	7.5	#4 @ 6.5"	#4 @ 10.5"	#5 @ 6"
4-6	7.5	#4 @ 6.5"	#4 @ 10"	#5 @ 6"
4-9	7.5	#4 @ 6"	#4 @ 10"	#5 @ 6"

5-0	7.5	#5 @ 9"	#4 @ 9.5"	#5 @ 6"
5-3	7.5	#5 @ 8.5"	#4 @ 9"	#5 @ 6.5"
5-6	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
5-9	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
6-0	7.5	#5 @ 7.5"	#4 @ 8"	#5 @ 6.5"
6-3	7.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 6.5"
6-6	7.5	#5 @ 7"	#4 @ 7.5"	#5 @ 6.5"
6-9	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-0	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-3	7.5	#5 @ 6.5"	#4 @ 6.5"	#5 @ 6.5"
7-6	7.5	#5 @ 6.5"	#5 @ 10"	#5 @ 6.5"
7-9	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-0	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-3	7.5	#5 @ 6"	#5 @ 9.5"	#5 @ 6.5"

Table 40.19-1

Reinforcing Steel for Deck Slabs on Girders for Deck Replacements – HS20 Loading

Max. Allowable Design Stresses: f_c' = 4000 psi, f_y = 60 ksi, Top Steel 2-1/2" Clear, Bottom Steel 1-1/2" Clear, Future Wearing Surface = 20 lbs/ft. Transverse and longitudinal bars shown in table are for one layer only. Place identical steel in both top and bottom layer, except in negative moment region. Use in top layer for slab on steel girders in negative moment region when not designed for negative moment composite action.

40.20 References

1. *A Study of Policies for the Protection, Repair, Rehabilitation, and Replacement of Concrete Bridge Decks* by P.D. Cady, Penn. DOT.

2. *Concrete Sealers for Protection of Bridge Structures, NCHRP Report 244*, December, 1981.

3. *Durable Bridge Decks* by D. G. Manning and J. Ryell, Ontario Ministry Transportation and Communications, April, 1976.

4. *Durability of Concrete Bridge Decks, NCHRP Report 57*, May, 1979.

5. *Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads, NCHRP Report 243*, December, 1981.

6. *Strength of Concrete Bridge Decks* by D. B. Beal, Research Report 89 NY DOT, July, 1981.

7. *Latex Modified Concrete Bridge Deck Overlays - Field Performance Analysis* by A. G. Bisharu, Report No. FHWA/OH/79/004, October, 1979.

8. *Standard Practice for Concrete Highway Bridge Deck Construction by ACI Committee 345, Concrete International*, September, 1981.

9. *The Effect of Moving Traffic on Fresh Concrete During Bridge Deck Widening* by H. L. Furr and F. H. Fouad, Paper Presented 61 Annual TRB Meeting, January, 1982.

10. *Control of Cracking in Concrete Structures by ACI Committee 224*, Concrete International, October, 1980.

11. *Discussion of Control of Cracking in Concrete Structures* by D. G. Manning, Concrete International, May, 1981.

12. *Effects of Traffic-Induced Vibrations on Bridge Deck Repairs, NCHRP Report 76*, December, 1981.

13. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary,* 2011.

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45.1 General

The 1967 collapse of the Silver Bridge in West Virginia prompted the development of the National Bridge Inspection Standards (NBIS) which require each State Highway Department of Transportation to inspect, prepare reports, and determine load ratings for bridge structures on all public roads. Soon after the development of the NBIS, supporting documents, including the FHWA Bridge *Inspector's Reference Manual* and the AASHTO *Manual for Condition Evaluation of Bridge*s were developed to help in implementing these standards.

In 2011, AASHTO released *The Manual for Bridge Evaluation (MBE)*. The manual replaced the earlier manuals: *The Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (AASHTO LRFR) and Manual for Condition Evaluation of Bridges*. Although the manual emphasizes the LRFR method, it also provides rating procedures for the Load Factor Rating (LFR) and Allowable Stress Rating (ASR) methodologies. For this reason, it will be the governing manual utilized by WisDOT for load rating structures.

Bridge load ratings are performed for specific purposes, such as: National Bridge Inventory (NBI) reporting, overweight permit load checks, bridge rehabilitation, etc. However, the main purpose of load rating is to determine the safe live load capacity of a structure. It would be convenient if some simple measure such as gross vehicle weight could be used to determine a bridge's capacity. However, the actual capacity depends on many factors, such as the gross vehicle weight, the axle configuration, the distribution of loads between the axles, etc. It is generally accepted that a bridge that can carry a given load on two axles can carry the same load or a larger load spread over several axles. Since it is not practical to rate a bridge for the nearly infinite number of axle configurations of trucks on our highways, bridges are rated for standard vehicles which are representative of the actual vehicles in use today. These standard vehicles will be discussed later in this document.

Whenever a bridge on the State Trunk Highway System is not able to safely carry the loads allowed by State Statute, it is load posted. Current Wisconsin State Statutes allow a gross vehicle weight of 80,000 pounds while loads up to 170,000 pounds are allowed for annual permit loads.

The FHWA currently requires that two capacity ratings, referred to as the Inventory Rating and Operating Rating be submitted with the NBI file. The Inventory Rating is the load level that a structure can safely sustain for an indefinite period. The Operating Rating is the absolute maximum permissible load level to which a structure may be subjected. The FHWA requires that the standard AASHTO HS truck or lane loading be used as the vehicle when load rating with the Load Factor Rating method (LFR) and the Allowable Stress Rating (ASR); and that the AASHTO HL-93 loading be utilized as the vehicle when load rating with the Load and Resistance Factor method (LRFR). A detailed explanation of each method, as well as a guide for when to utilize each method can be found in Section 45.3.

Bridges being analyzed for staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFR (or LFR or ASR, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

Note: Bridges shall be load rated for any project that results in a change to the loading. This requirement includes deck replacements, deck overlays (new overlays, removal of existing overlays and replacement, polymer overlays, etc.), bridge widenings, superstructure replacements/moved girders, etc.

45.2 Bridge Inspections

To determine the strength or load carrying capacity of a bridge, it is necessary to have a complete description of the bridge. This can include as-built plans, repair records, photographs, design/rating calculations, and current inspection information. If drawings are not available or are incomplete, they must be reproduced by means of complete measurements taken in the field. The present condition can be gathered from recent field inspection reports.

Inspections of bridges on the State Trunk Highway Network are performed by trained personnel from the Regional Maintenance Sections utilizing guidelines established in the latest edition of the *WisDOT Structure Inspection Manual*. Engineers from the Bureau of Structures may assist in the inspection of bridges with unique structural problems or when it is suspected that a reduction in load capacity is warranted. To comply with the National Bridge Inspection Standards (NBIS), it is required that all bridges on Federal Aid Routes be routinely inspected at intervals not to exceed two years. More frequent inspections are performed for bridges which are posted for load capacity or when it is warranted by their condition. In addition, special inspections such as underwater diving or fracture critical are performed when applicable.

Inspectors enter inspection information into the Highway Structures Information System (HSI), an online Bridge Management System developed by WisDOT. This database is used to create the NBI file and is also the central source for documents such as plans, maintenance records, design calculations and rating calculations that are critical when calculating structural ratings.

HSI also supplies a “re-rate flag” on the bridge inspection form that allows the inspector to schedule a structure for rating analysis if field conditions dictate the need. This flag can be queried by owners to obtain a quick list of structures needing analysis at the end of the inspection cycle. Ratings for State Owned Structures are generally performed by Bureau of Structures staff. Load Ratings for Local Owners are the responsibility of the owner.

45.2.1 Condition of Bridge Members

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects for the capacity when shear or moment is chosen for use in the basic rating equation.

As mentioned above, the rating of an older bridge for its load-carrying capacity should be based on a current field inspection. All physical features of a bridge which have an effect on the structural integrity should be examined. The inspector should note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, the inspector should make a determination of the loss in a cross-sectional area as closely as reasonably possible. They should also determine if deep pits, nicks or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities may be necessary if such conditions exist.

If not otherwise noted in the plans, inspectors should note the size, number, and the relative location of bolts and rivets through tension members so that the net area of the section can be calculated. Any misalignment, bends, or kinks in compression members should also be measured carefully. Such defects will have a great effect on the load-carrying capability of a member and may be the controlling factor in the load-carrying capacity of the entire structure. Also, the inspector should examine the connections of compression members carefully to see if they are detailed such that eccentricities are introduced which must be considered in the structural analysis.

The effective area of members to be used in the calculations shall be the gross area less that portion which has deteriorated due to decay or corrosion. The effective area should be adjusted for rivet or bolt holes in accordance with the AASHTO LRFD Design Specifications or the AASHTO Standard Specifications, where applicable.

45.3 Load Rating Methodologies

There are two primary methods of load rating bridge structures that will be utilized by WisDOT. Both methods are detailed in the AASHTO *MBE*. They are as follows:

- Load Factor Rating (LFR)
- Load and Resistance Factor Rating (LRFR)

LFR has been used since the early 1990's to load rate bridges in Wisconsin. The basic philosophy behind this method is to assign factors of safety to both dead and live loads. Loads that are more predictable, such as dead loads, are assigned a lower factor of safety while loads that are less predictable, such as truck loads, are assigned a higher factor of safety. The rating is determined such that the effect of the factored loads does not exceed the strength of the member. LFR shall be utilized on all structures not constructed of timber in the WisDOT inventory designed with either LFD or ASD and is thoroughly covered in Section 45.3.3. A detailed description of this method can also be found in **MBE [6B]**.

LRFR employs the same basic principles as LFR for the load factors, but also utilizes resistance factors to account for uncertainties in member condition, material properties, etc. This methodology shall be used for all structures designed using Load and Resistance Factor Design (LRFD). This method is covered in Section 45.3.2. A detailed description of this method can be found in **MBE [6A]**.

The load rating of timber structures shall follow a third method as follows:

- Allowable Stress Rating (ASR)

ASR has been used to load rate timber bridges in Wisconsin and should be used for all timber structures not designed by Load and Resistance Factor Design. The basic philosophy behind this method assigns an appropriate factor of safety to the limiting stress of the material being analyzed. The maximum stress in the member produced by actual loadings is then checked for sufficiency. This method is covered in Section 45.3.4. A detailed description of this method can also be found in **MBE [6B]**.

WisDOT policy item:

Rate the bridge using LFR provided it was designed using ASD or LFD, unless it is a timber structure. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the BOS Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

45.3.1 General Assumptions

The following concepts shall be applied to the load rating of structures in Wisconsin:

- The AASHTO *MBE* has provisions for LRFR, LFR and ASR and shall be the main load rating manual for WisDOT. This chapter serves as a supplement to the *MBE* and deals primarily with WisDOT exceptions, interpretations, and policy decisions.

- Substructures generally do not control the load rating. Therefore, a complete analysis of the substructure is not required if, in the judgment of the load rating engineer, the substructure has greater capacity than the superstructure. Scenarios where substructure element conditions may prompt a load rating include, but are not limited to:

  - Extensive section loss from corrosion or rot (common in timber and steel piles)

  - Scour, undermining, or settlement which may affect a footing's bearing capacity or a column's unbraced length

  - Collision/impact damage

  - Substructure components with deterioration and lack of redundancy

- Reinforced concrete bridge decks on redundant, multi-girder bridges need not be rated unless damage, deterioration, or other concerns merit this analysis, as determined by the judgment of the load rating engineer.

- Dead loads shall be distributed as described in 17.2.7 for slabs and 17.2.8 for slab on girders.

- Live loads shall be distributed as described in 17.2.7 or 18.4.5.1 for slabs and 17.2.8 for slab on girders.

- When performing a rating on a bridge with a steel superstructure element (deck girder, floorbeam, or stringer) carrying a concrete deck, the element should be assumed to have full composite action if it was designed for composite action and it has shear studs or angles that are spaced at no more than 2'-0" centers.

- For continuous girder type bridges, the negative moment steel shall conservatively consist of only the top mat of steel over the piers.

- If there are no plans for a bridge with a steel superstructure carrying a concrete deck, it shall be assumed to be non-composite for purposes of load rating unless there is sufficient documentation to prove that it was designed for composite action and that shear studs or angles were used in the construction.

- The governing rating shall be the lesser of the Strength Limit States (strength and stability, both local and global, provided to resist the specified load combinations that a bridge is expected to experience in its design life) and/or Service Limit States (restrictions on stress, deformation, and crack width under regular service conditions) of the critical component. Note: Prestressed concrete bridges designed during the 1960s and 1970s may not meet current shear capacity requirements. If shear capacities are determined to be insufficient, the responsible engineer should contact the Bureau of Structures Development Section Rating Unit. If the load rating engineer, utilizing engineering judgment, determines that certain components will not control the rating, then a full analysis of the non-controlling element is not required.

- For prestressed girder type bridges, elastic gains shall be neglected for a conservative approach.

45.3.2 Load and Resistance Factor Rating (LRFR) Method

All bridge structures designed utilizing Load and Resistance Factor Design (LRFD) shall be rated LRFR per the 2011 AASHTO *Manual for Bridge Evaluation.* The basic rating equation, per **MBE [Equation 6A.4.2.1-1]**, is:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

For the Strength Limit States:

$$C = \phi_C \phi_S \phi R_n$$

Where the following lower limit shall apply:

$$\phi_C \phi_S \geq 0.85$$

Where:

RF	=	Rating factor
C	=	Capacity
R_n	=	Nominal member resistance
DC	=	Dead-load effect due to structural components and attachments
DW	=	Dead-load effect due to the wearing surface and utilities
P	=	Permanent loads other than dead loads
LL	=	Live load effects
IM	=	Dynamic load allowance
γ_{DC}	=	LRFR load factor for structural components and attachments
γ_{DW}	=	LRFR load factor for wearing surfaces and utilities
γ_P	=	LRFR load factor for permanent loads other than dead loads = 1.0

γ_{LL} = LRFR evaluation live load factor

ϕ_c = Condition factor

ϕ_s = System factor

ϕ = LRFR resistance factor

The LRFR methodology is comprised of three distinct procedures:

- Design Load Rating (first level evaluation)
- Legal Load Rating (second level evaluation)
- Permit Load Rating (third level evaluation)

The results of each procedure serve specific uses and also guide the need for further evaluations to verify bridge safety or serviceability. A flow chart outlining this approach is shown in Figure 45.3-1. The procedures are structured to be performed in a sequential manner, as needed, starting with the Design Load Rating. Load rating for AASHTO legal loads is only required when a bridge fails (RF < 1) the Design Load Rating at the Operating level.

Note that when designing a new structure, it is required that the rating factor be greater than one for the HL-93 vehicle at the Inventory Level; therefore, a Legal Load Rating will never be required on a newly designed structure.

Similarly, only bridges that pass the Legal Load Rating (RF ≥ 1) can be evaluated utilizing the Permit Load Rating procedures. This level is used for both the Wisconsin Standard Permit Vehicle Design Check, as discussed in Section 45.6 and for Single Trip permit evaluation as discussed in Section 45.7.3.

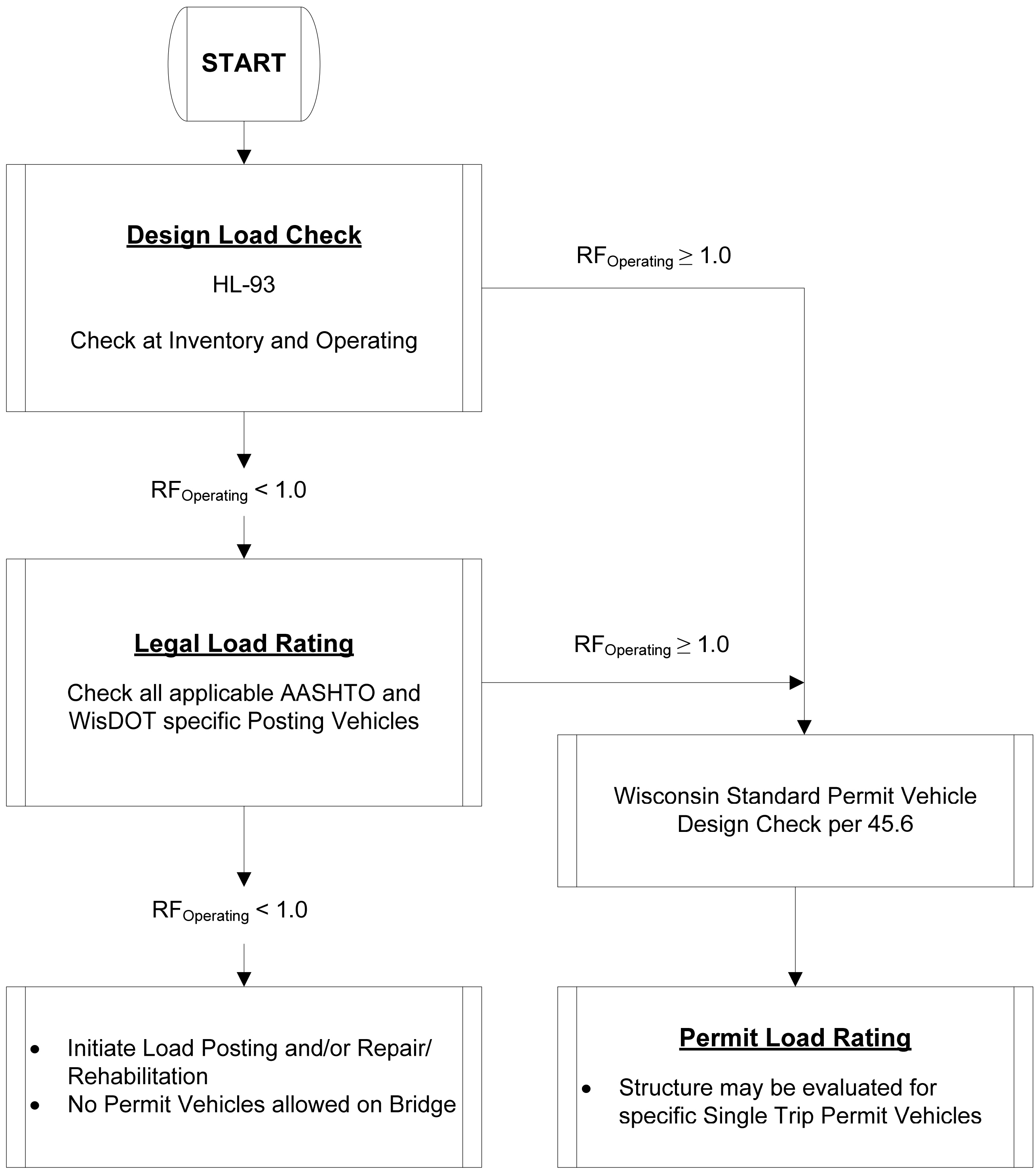


Figure 45.3-1
Load and Resistance Factor Rating Flow Chart

45.3.2.1 Limit States

Strength I is used for the ultimate capacity of structural members and is the primary limit state utilized by WisDOT for load rating. Service limit states are utilized to limit stresses, deformations, and crack widths under regular service conditions and are often considered optional in load rating calculations. Service limit states checks that are considered optional are shaded in Table 45.3-1.

45.3.2.2 Load Factors

The load factors for the Design Load Rating shall be taken as shown in Table 45.3-1. The load factors for the Legal Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-2. The load factors for the Permit Load Rating shall be taken as shown in Table 45.3-1 and Table 45.3-3. Again, note that the shaded values in Table 45.3-1 indicate optional checks that are currently not required by WisDOT.

Bridge Type	Limit State	Dead Load DC	Dead Load DW	Design Load		Legal Load	Permit Load
				Inventory	Operating		
				LL	LL	LL	LL
Steel	Strength I	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service I	1.00	1.00	--	--	--	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3
	Service III	1.00	1.00	0.80	--	1.00	--
	Service I	1.00	1.00	--	--	--	1.00
Timber	Strength I	1.25	1.50	1.75	1.35	Table 45.3-2	Table 45.3-3

Table 45.3-1
Limit States and Load Factors for LRFR

Loading Type	Live Load Factor
AASHTO Legal Vehicles, State Specific Vehicles, and Lane Type Legal Load Models	1.45
Specialized Haul Vehicles (SU4, SU5, SU6, SU7)	1.45

Table 45.3-2
Live Load Factors for Legal Loads in LRFR

Permit Type	Loading Condition	Distribution Factor	Live Load Factor
Annual	Mixed with Normal Traffic	Two or more lanes	1.30
Single Trip	Mixed with Normal Traffic	One Lane	1.20
Single Trip	Escorted with no other vehicles on the bridge	One Lane	1.10

Table 45.3-3
Permit Load Factors for LRFR

45.3.2.3 Resistance Factors

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance. Resistance factors for concrete and steel structures are presented in 17.2.6, and resistance factors for timber structures are presented in **MBE [6A.7.3]**.

45.3.2.4 Condition Factor: ϕ_C

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles. Current WisDOT policy is to set this factor equal to 1.0.

45.3.2.5 System Factor: ϕ_S

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factor member capacities reduced, and, accordingly, will have lower ratings. The aim of the system factor is to provide reserve capacity for safety of the traveling public. See Table 45.3-4 for WisDOT system factors.

Superstructure Type	ϕ_S
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing ≤ 6.0 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4.0 ft	0.95
All Other Girder and Slab Bridges	1.00
Floorbeam Spacings > 12.0 ft and Non-Continuous Stringers	0.85
Redundant Stringer Subsystems Between Floorbeams	1.00

Table 45.3-4
System Factors for WisDOT

45.3.2.6 Design Load Rating

The Design Load Rating assesses the performance of bridges utilizing the LRFD design loading (HL-93). This serves as a screening process to identify bridges that should be load rated for legal loads. If a structure has a RF ≥ 1.0 at the Operating level for HL-93, proceeding to the Legal Load Rating is not required. However, the load rating engineer is still required to rate the Wisconsin Standard Permit Vehicle as shown in Section 45.6. The Design Load Rating produces Inventory and Operating level rating factors for the HL-93 Loading. The results are used to develop the NBI file. Note that when designing a new structure, it is required that the RF > 1 at the Inventory Level.

45.3.2.6.1 Design Load Rating Live Load

The LRFD design live load, HL-93, shall be utilized as the rating vehicle. The components of the HL-93 loading are described in 17.2.4.2.

45.3.2.7 Legal Load Rating

Bridges that do not satisfy the HL-93 Design Load Rating check (RF < 1.0 @ Operating) should be evaluated for legal loads to determine if posting or strengthening of the structure is required. For more information on the load posting of bridges, see Section 45.4.

45.3.2.7.1 Legal Load Rating Live Load

The live loads used for Legal Load Rating calculations shall be as described in Section 45.4.1.

45.3.2.7.2 Legal Load Rating Load Posting Equation

When the rating factor (RF) calculated for each legal truck described above is greater than 1.0, the bridge does not need to be posted. When for any legal truck the RF is between 0.3 and 1.0, then the following equation should be used to establish the safe posting load for that vehicle (see **MBE [Equation 6A8.3-1]**):

$$\text{Posting} = \frac{W}{0.7}\left[(RF) - 0.3\right]$$

Where:

RF = Legal load rating factor

W = Weight of the rating vehicle

When the RF for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the bridge. If necessary, the structure may need to be closed until it can be repaired, strengthened, or replaced. This formula is only valid for LRFR load posting calculations.

45.3.2.8 Permit Load Rating

This level of load rating serves many purposes for WisDOT. First, it is the level of load rating analysis required for all structures when performing the Wisconsin Standard Permit Vehicle Design Check as illustrated in Section 45.6. Second, this level is used, whenever necessary, for issuance of Single Trip permits. As their name indicates, single trip permits are valid for only one trip. Each single trip permit vehicle is analyzed for every structure it will cross.

45.3.2.8.1 Permit Load Rating Live Load

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (Figure 45.6-1). Specifics on this analysis can be found in Section 45.6.

For specific Single Trip permit applications, the actual truck configuration described in the permit shall be the live load used to analyze all pertinent structures.

WisDOT policy items:

WisDOT interpretation of **MBE [6A.4.5.4.1]** is for spans up to 200'-0", only the permit vehicle shall be considered present in the lane. For spans 200'-0" in length or greater an additional lane load shall be applied to simulate closely following vehicles. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the permit load effects.

Also note, as stated in the footnote of **MBE [Table 6A.4.5.4.2a-1]**, when using a single-lane LRFD distribution factor, the 1.2 multiple presence factor should be divided out from the distribution factor equations.

45.3.3 Load Factor Rating (LFR) Method

All bridge structures designed utilizing LFD or ASD, other than timber structures, shall be rated (possible exception stated in Policy Item under Section 45.3) utilizing LFR per the AASHTO *Manual for Bridge Evaluation* (**MBE [6B]**). The basic rating equation can be found in **MBE [Equation 6B.4.1-1]** and is:

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)}$$

Where:

RF	=	The rating factor for the live load carrying capacity
C	=	The capacity of the member

D = The dead load effect on the member

L = The live load effect on the member

I = The impact factor to be used with the live load effect

A_1 = Factor for dead loads

A_2 = Factor for live load

45.3.3.1 Live Loads

Similar to LRFR, there are three potential checks to be made in LFR that are detailed in the flow chart shown in Figure 45.3-2. For purposes of calculating the Inventory and Operating rating of the structure, the live load to be used should be the HS20 truck or lane loading as shown in Figures 17.2-1 and 17.2-3. For conducting the Wisconsin Standard Permit Vehicle Design Check, use the loading shown in Figure 45.6-1. For determination of postings, refer to 45.4.1 for the proper posting vehicles.

One important item to note: when rating permit loads for continuous concrete slab bridges of 30’-0” width or more wheel loads are distributed over a width of 12’-0”, which is a simplified adaptation of the distribution factor in the Ontario Bridge Design Code.

45.3.3.2 Load Factors

See Table 45.3-5 for load factors to be used when rating with the LFR method. The nominal capacity, C, is the same regardless of the rating level desired.

LFR Live Load Factors		
Rating Level	A_1	A_2
Inventory	1.3	2.17
Operating	1.3	1.3

Table 45.3-5
LFR Live Load Factors

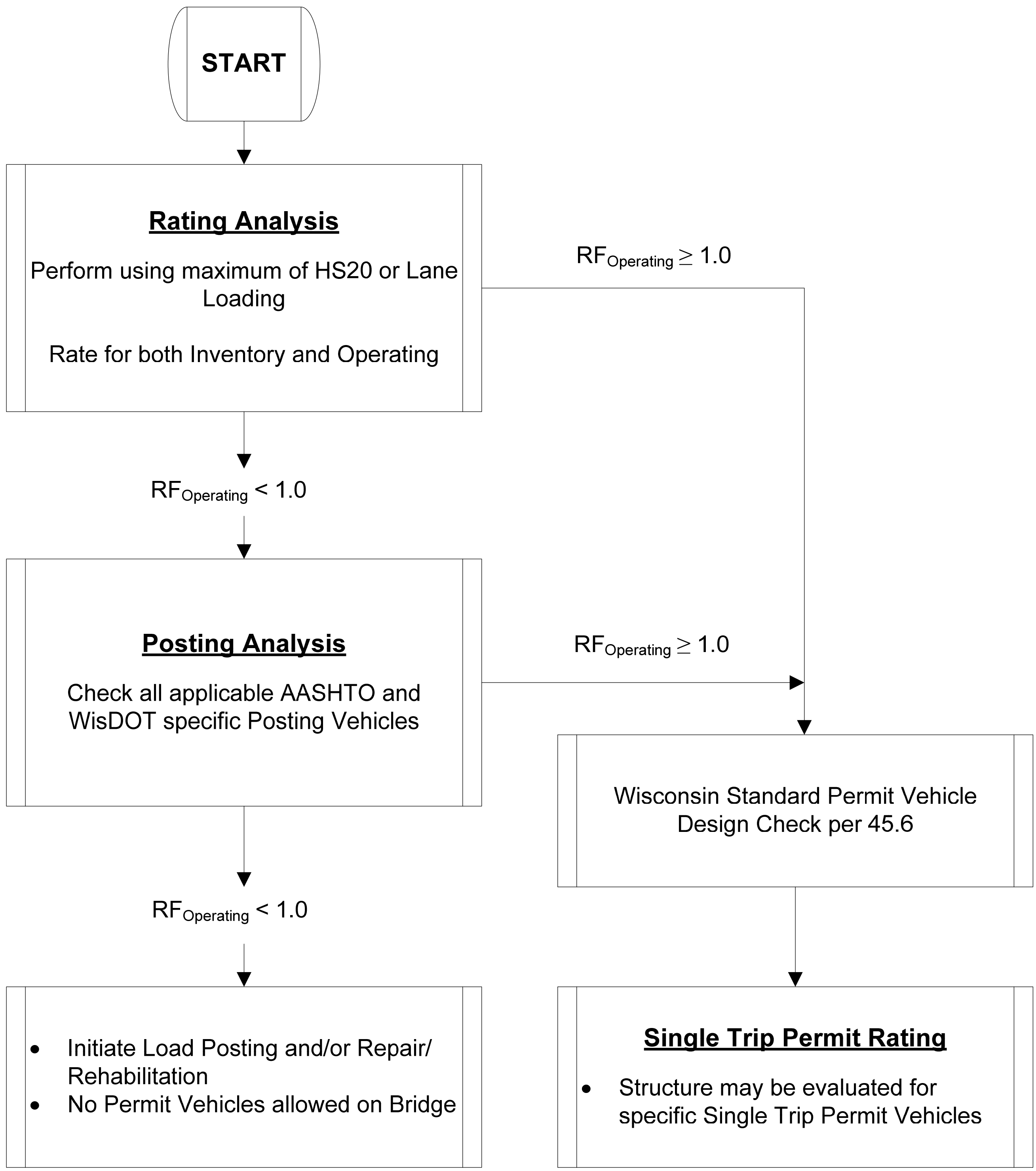


Figure 45.3-2
Load Factor Rating and Allowable Stress Rating Flow Chart

45.3.4 Allowable Stress Rating (ASR) Method

All timber bridge structures designed utilizing ASD shall be rated utilizing ASR per the AASHTO *Manual for Bridge Evaluation* (**MBE [6B]**). The basic rating equation can be found in **MBE [Equation 6B.4.1-1]** and is:

$$RF = \frac{C - D}{L(1 + I)}$$

Where:

RF	=	The rating factor for the live load carrying capacity
C	=	The capacity of the member
D	=	The dead load effect on the member
L	=	The live load effect on the member
I	=	The impact factor to be used with the live load effect

45.3.4.1 Live Loads

Similar to LRFR and LFR, there are three potential checks to be made in ASR. For purposes of calculating the Inventory and Operating rating of the structure, the live load to be used should be the HS20 truck. For conducting the Wisconsin Standard Permit Vehicle Design Check, use the loading shown in Figure 45.6-1. For determination of postings, refer to 45.4.1 for the proper posting vehicles.

45.4 Bridge Posting

A bridge should be capable of carrying a minimum gross live load weight of three tons at the Inventory level. Bridges not capable of carrying a minimum gross live load weight of three tons at the Operating level must be closed. When deciding whether to close or post a bridge, consider the volume of traffic, the character of traffic, the likelihood of overweight vehicles and the enforceability of weight posting.

In certain cases, a concrete bridge need not be posted for restricted loading when it has been carrying its design level traffic for an appreciable length of time and shows no distress. This general rule may apply to bridges for which details of the reinforcement are not known but it should be used with caution. In 1974, the *AASHTO Interim Specifications Bridges* made a number of significant changes in the design of reinforced concrete. Load factor design, bar steel development lengths and elimination of the old bond stress concept were some of the changes. One reason for these changes was to make sure that an overloaded concrete structure failed by yielding of the reinforcing in bending and not by a sudden concrete shear or bond failure. Thus, concrete bridges designed prior to 1974, when approaching their ultimate loading, may not exhibit a ductile failure (i.e. bending failure). However, if the load rating engineer chooses not to post, the structure shall be inspected at a interval not to exceed six months for signs of distress until such time as the bridge is either strengthened or replaced. In lieu of frequent inspections or posting, a bridge may be load tested to determine its capacity.

The State Bridge Maintenance Engineer has the authority to post a bridge and must issue the approval to post any State bridge.

WisDOT policy items:

Multiple lane distribution factors using operating load factors are used for determining bridge capacities for posting and annual permits for bridge widths 18'-0" or larger. Single lane distribution factors using operating load factors are used for bridge widths less than 18'-0" and for single trip permits.

However, for specialized annual permit vehicles in Figure 45.4-3, always use a single lane distribution factor and an operating load factor, regardless of bridge width.

A bridge is posted for the lowest restricted weight limit of the standard posting vehicles. If the RF $\geq$ 1.0 for a given vehicle at the operating level, then a posting will not be required for that particular vehicle. If the RF < 1.0 for a given vehicle at the operating level, then the bridge shall be posted. To calculate the capacity, in tons, on a bridge for a given posting vehicle utilizing LFR, multiply the RF by the gross vehicle weight in tons. To calculate the posting load for a bridge analyzed with LRFR, refer to 45.3.2.7.2.

Also, for State Trunk Highway Bridges, current WisDOT policy is to post structures with a Wisconsin Standard Permit Vehicle (Wis-SPV) rating of 120 kips or less utilizing a single lane distribution factor.

When the lane-type load model (see Figure 45.4-4) governs the load rating, the equivalent truck weight for use in calculating a safe load capacity for the bridge shall be taken as 80 kips as is specified in **MBE [6A.4.4.4]**.

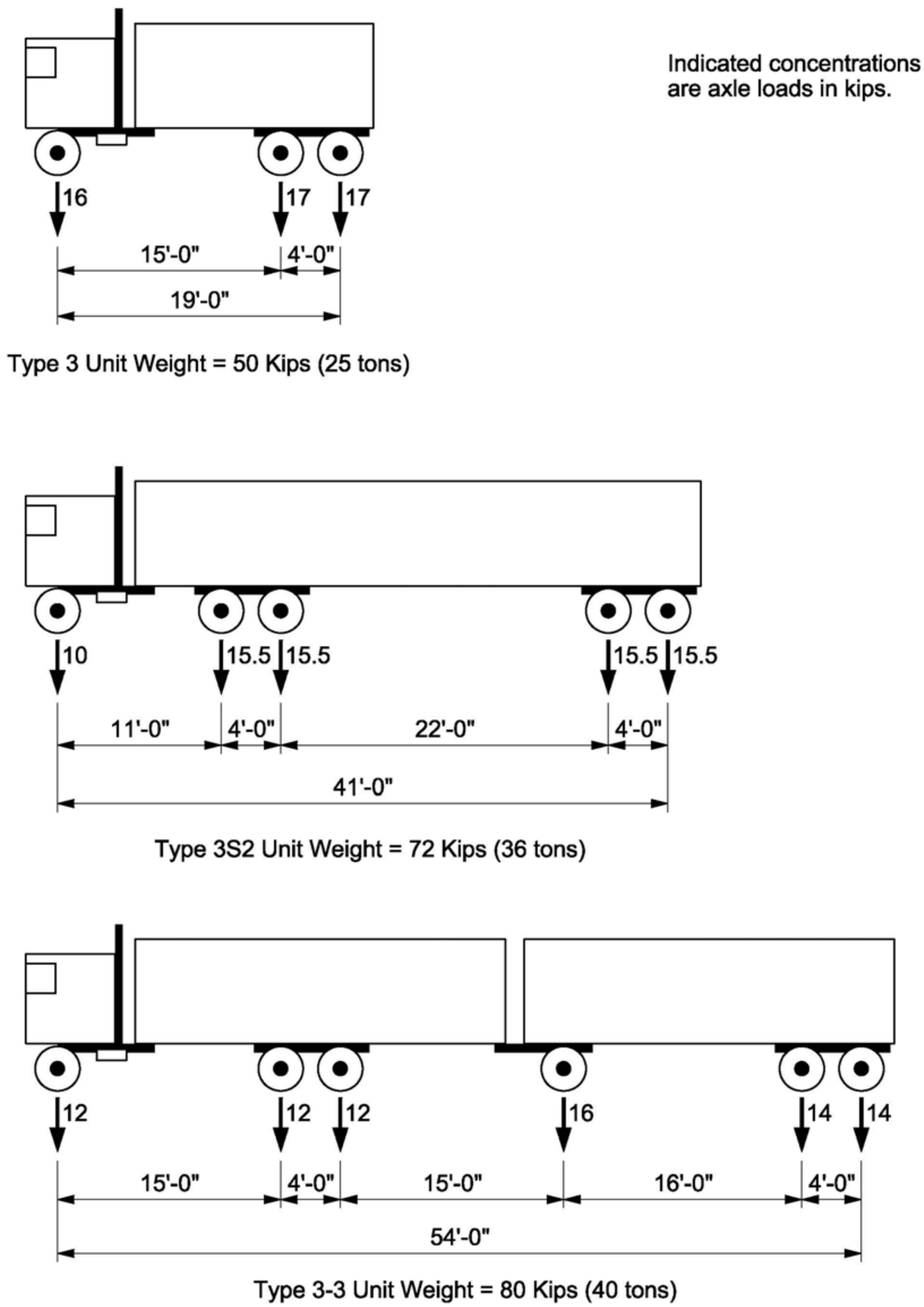


Figure 45.4-1
AASHTO Commercial Vehicles

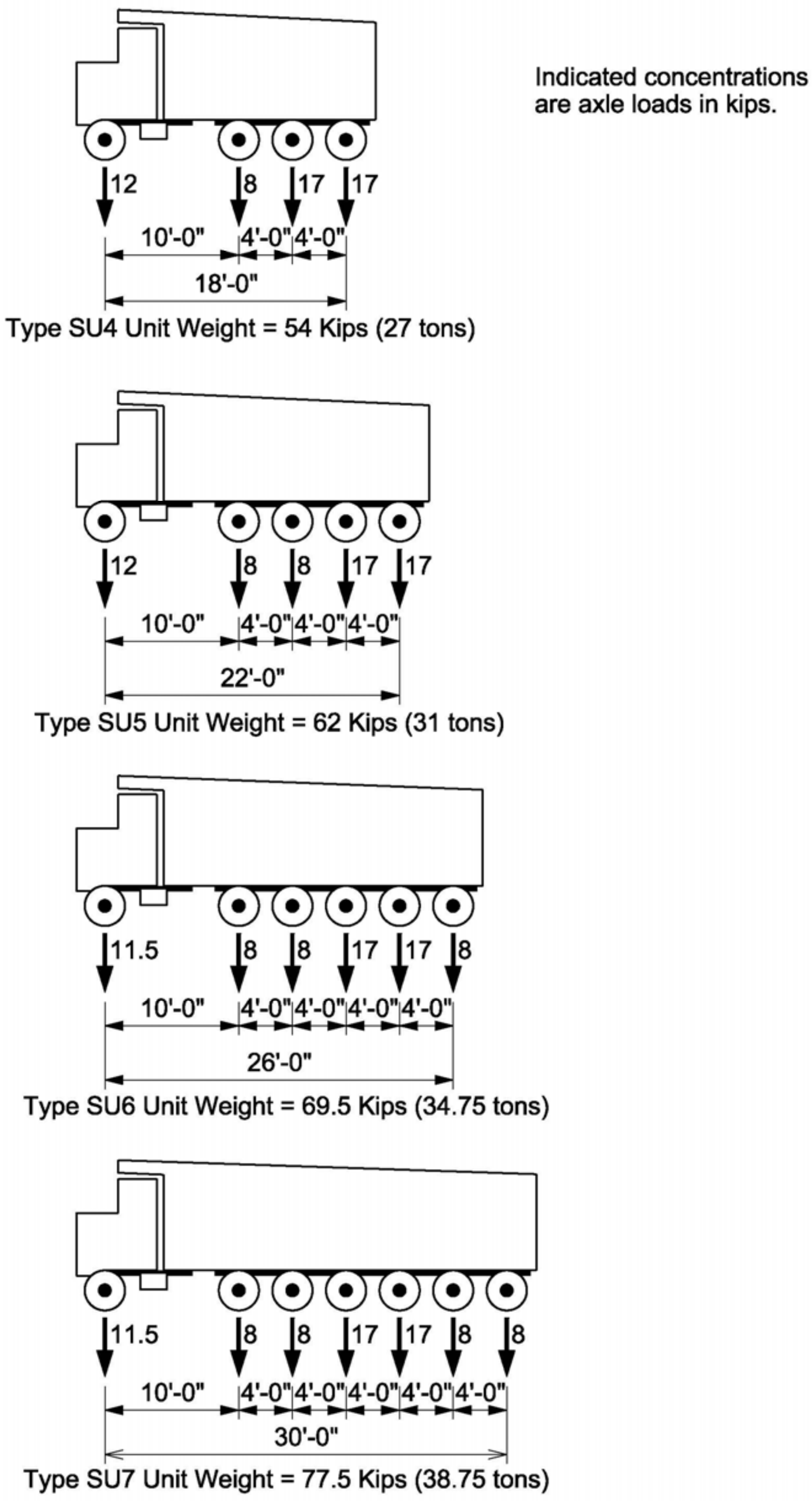


Figure 45.4-2

AASHTO Specialized Hauling Vehicles

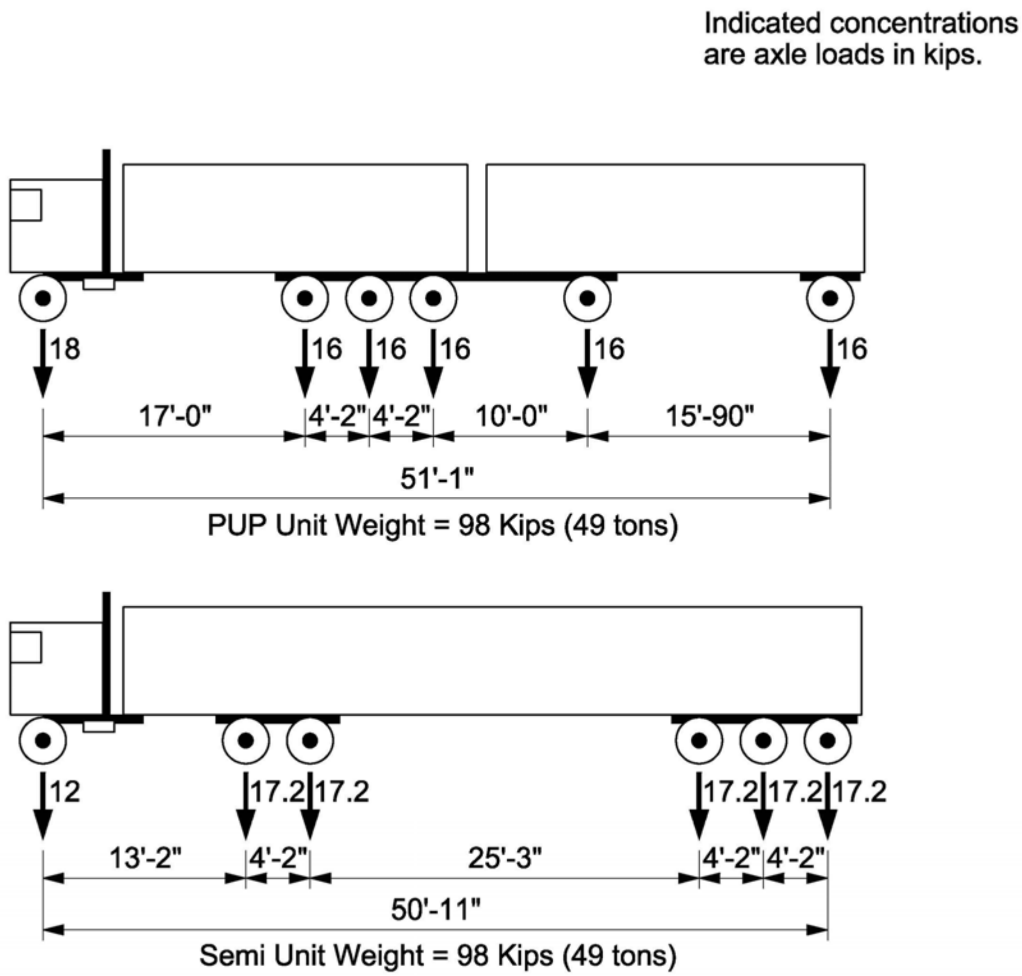


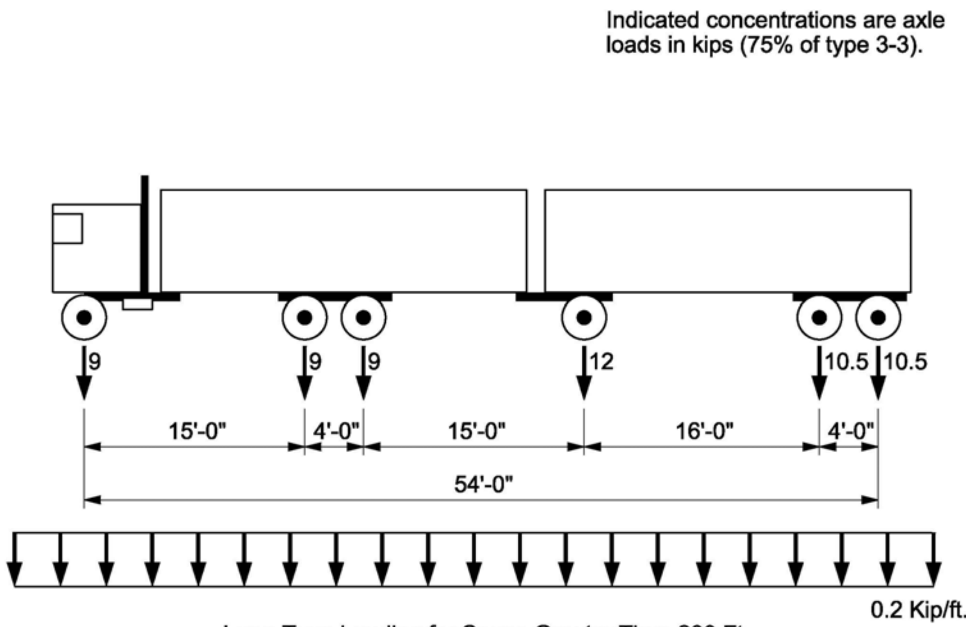
Figure 45.4-3
WisDOT Specialized Annual Permit Vehicles

45.4.1 Posting Live Loads

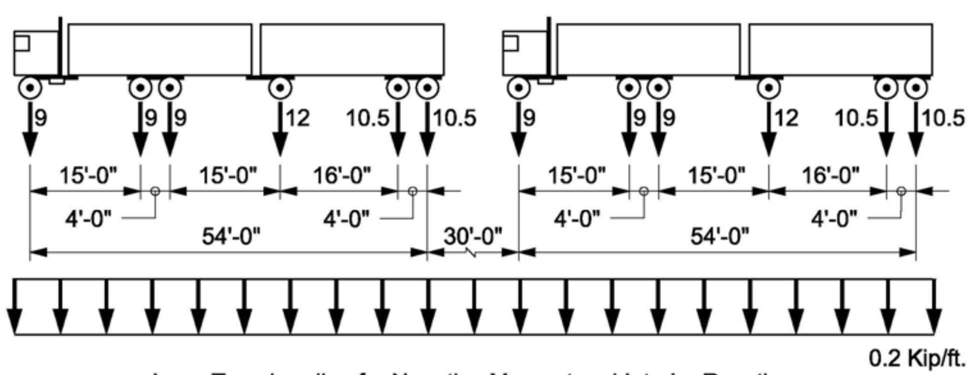
The live load to be used in the rating formula for posting considerations should be any of the three typical AASHTO Commercial Vehicles (Type 3, Type 3S2, Type 3-3) shown in Figure 45.4-1, any of the four AASHTO Specialized Hauling Vehicles (SU4, SU5, SU6, SU7) shown in Figure 45.4-2, the Wisconsin Standard Permit Vehicle shown in Figure 45.6-1, or in certain cases the specialized annual permit vehicles shown in Figure 45.4-3.

As stated in **MBE [6A.4.4.2.1a]**, for spans up to 200’, only the vehicle shall be considered present in the lane for positive moments. It is unnecessary to place more than one vehicle in a lane for spans up to 200’ because the load factors provided have been modeled for this possibility. For spans 200’ in length or greater, the AASHTO Type 3-3 truck multiplied by 0.75 shall be analyzed combined with a lane load as shown in Figure 45.4-4. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the vehicle load effects.

Also, for negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 trucks multiplied by 0.75 shall be used. The trucks should be heading in the same direction and should be separated by 30 ft as shown in Figure 45.4-4. There are no span length limitations for this negative moment requirement.



Lane-Type Loading for Spans Greater Than 200 Ft.



Lane-Type Loading for Negative Moment and Interior Reaction.

Figure 45.4-4

Lane Type Legal Load Models

45.4.2 Posting Signage

Current WisDOT policy is to post State bridges for only one tonnage capacity. Bridges which cannot carry the maximum weight for the vehicles described in Section 45.4.1 using Operating Rating criteria are posted with one of the standard signs, shown in Figure 45.4-5 showing the bridge capacity for the governing vehicle, which should conform to the requirements of the *Manual for Uniform Traffic Control Devices (MUTCD)*.

In the past, local bridges were occasionally posted with the signs shown in Figure 45.4-6 using the H20, Type 3 and Type 3S2 vehicles. The H20 represented the two-axle vehicle, the Type 3 represented the three-axle vehicle and the Type 3S2 represented the combination vehicle. This practice is not encouraged by WisDOT and is generally not allowed for State owned structures, except with permission from the State Bridge Engineer.

WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

Figure 45.4-5
Standard Signs Used for Posting Bridges

WEIGHT LIMIT 2 AXLE VEHICLES 15 TONS 3 AXLE VEHICLES 20 TONS COMBINATION VEHICLES 30 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS

Figure 45.4-6
Historic Load Posting Signs

45.5 Material Strengths and Properties

Material properties shall be as stated in AASHTO *MBE* or as stated in this chapter. Often when rating a structure without a complete set of plans, material properties are unknown. The following section can be used as a guideline for the rating engineer when dealing with structures with unknown material properties. If necessary, material testing may be needed to analyze a structure.

45.5.1 Reinforcing Steel

The allowable unit stresses and yield strengths for reinforcing steel can be found in Table 45.3-1. When the condition of the steel is unknown, they may be used without reduction. Note that Wisconsin started to use Grade 40 bar steel about 1955-1958; this should be noted on the plans.

Reinforcing Steel Grade	Inventory Allowable (psi)	Operating Allowable (psi)	Minimum Yield Point (psi)
Unknown	18,000	25,000	33,000
Structural Grade	19,800	27,000	36,000
Grade 40 (Intermediate)	20,000	28,000	40,000
Grade 60	24,000	36,000	60,000

Table 45.5-1
Yield Strength of Reinforcing Steel

45.5.2 Concrete

The following are the maximum allowable unit stresses in concrete in pounds per square inch (see Table 45.5-2). Note that the "Year Built" column may be used if concrete strength is not available from the structure plans.

Year Built	Inventory Allowable (psi)	Operating Allowable (psi)	Compressive Strength (F'c) (psi)	Modular Ratio
Before 1959	1000	1500	2500	12
1959 and later	1400	1900	3500	10
For all non-prestressed slabs 1975 and later	1600	2400	4000	8
Prestressed girders before 1964 and all prestressed slabs	2000	3000	5000	6
1964 and later for prestressed girders	2400	3000	6000	5

Table 45.5-2
Minimum Compressive Strengths of Concrete

45.5.3 Prestressed Steel Strands

Table 45.5-3 contains values for uncoated Seven-Wire Stressed-Relieved and Low Relaxation Strands:

Year Built	Grade	Nominal Diameter of Strand (In)	Nominal Steel Area of Strand (In^2)	Yield Strength (psi)	Breaking Strength (psi)
Prior To 1963	250	$^{7}/_{16}$ (0.438)	0.108	213,000	250,000
Prior To 1963	250	½ (0.500)	0.144	212,500	250,000
1963 To Present	270	½ (0.500)	0.153	229,000	270,000
1973 To Present	270 Low Relaxation	½ (0.500)	0.153	242,500	270,000
1995 to Present	270 Low Relaxation	$^{9}/_{16}$ (0.600)	0.217	242,500	270,000

Table 45.5-3
Tensile Strength of Prestressing Strands

The “Year Built” column is for informational purposes only. The actual diameter of strand and grade should be obtained from the structure plans. If an option is given on the structure plans to use either stress relieved or low relaxation strand, or $^{7}/_{16}$” or $^{1}/_{2}$" diameter strand, consult the shop drawings for the new structure to see which option was exercised. If the shop drawings are not available, assume the option which gives the lowest operating rating was used.

45.5.4 Structural Steel

The **MBE [Table 6B.5.2.1-1]** gives allowable stresses for steel based on year of construction or known type of steel. For newer bridges refer to AASHTO design specifications.

Steel Type		AASHTO Designation	ASTM Designation	Minimum Tensile Strength, Fu (psi)	Minimum Yield Strength, Fy (psi)
Unknown Steel	Built prior to 1905			52,000	26,000
	1905 to 1936			60,000	30,000
	1936 to 1963				33,000
	After 1963				36,000
Carbon Steel		M 94 (1961)	A 7 (1967)	60,000	33,000
Nickel Steel		M 96 (1961)	A 8 (1961)	90,000	55,000
Silicon Steel	up to 1-1/8" thick	M 95 (1961)	A 94	75,000	50,000
	1-1/8" to 2" thick		A 94	72,000	47,000
	2" to 4" thick		A 94 (1966)	70,000	45,000
Low Alloy Steel			A441	75,000	50,000

Table 45.5-4
Minimum Yield Strength Values for Common Steel Types

45.6 Wisconsin Standard Permit Vehicle Design Check

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed. For LRFR bridge analysis, the requirements of Section 45.3.2.8.1 for lane loading shall be considered along with the Wis-SPV, where applicable.

When performing this design check for the Wis-SPV, the vehicle shall be evaluated for single-lane (single trip permit) distribution assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. For this rating, future wearing surface shall be considered. The engineer shall check to ensure the design (new or rehabilitation) has a minimum capacity to carry a gross vehicle load of 190 kips. Load distribution for this check is based on the interior strip or interior girder and the distribution factors given in 17.2.7, 17.2.8, or 18.4.5.1 where applicable.

If this check fails, then the engineer is required to adjust the design until the bridge can safely handle a minimum gross vehicle load of 190 kips.

Also note, as stated in the footnote of **MBE [Table 6A.4.5.4.2a-1]**, when using a single-lane LRFD distribution factor, the 1.2 multiple presence factor should be divided out from the distribution factor equations.

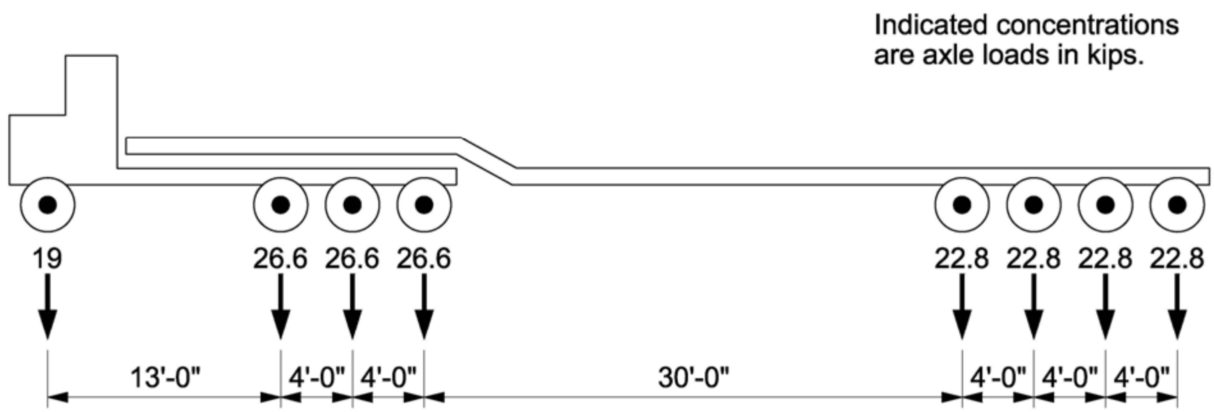


Figure 45.6-1
Wisconsin Standard Permit Vehicle (Wis-SPV)

45.7 Overweight Trip Permits

45.7.1 General Information

The load effects produced by the Wis-SPV were designed to completely envelope effects produced by all possible annual permit vehicle configurations. In addition, the Wis-SPV attempts to represent the truck most frequently used to carry loads requiring a single trip permit. However, in the case of single trip permits, each bridge on a State route is analyzed for the vehicle submitted by the trucking company prior to issuance of the specific permit, so it is not necessary, or feasible, for the Wis-SPV to envelope all possible single trip permit vehicles.

For overweight trip permit analysis, load distribution is based on the distribution factors given in 17.2.7, 17.2.8 or 18.4.5.1, where applicable. The analysis is done at the operating rating level.

45.7.2 Annual Trip Permit Information

Annual permits are only allowable for non-divisible loads such as machines, self-propelled vehicles, mobile homes, etc. They are usually valid for unlimited trips over the period of one year. The permit vehicle may mix in the traffic stream and move at normal speeds without any restrictions. Multi-lane distribution is used in the analysis.

The maximum annual permit weight is 170,000 lbs. and is subject to the axle weight limitations specified in Table 45.7-1.

Axle Configuration	Load (Pounds)
Single Axle	20,000 (2 Tires)
Single Axle	30,000 (3 Tires)
2-Axle Tandem	55,000
3-Axle Tandem	70,000
4-Axle Tandem	80,000

Table 45.7-1
Allowable Axle Weights for Annual Permits

A tandem axle is considered to be any group of two, three or four axles in which the centers of successive axles of the group are between 3’-6” and 6’-0”. If the spacing between any combination of single axles or tandem axle groups is less than 18’-0” the gross load of the combinations must be reduced. There is a length limitation of 50’-0” for single vehicles and 75’-0” for vehicle combinations.

Refer to the Division of Motor Vehicles (DMV) website for more information.

http://www.dot.wisconsin.gov/business/carriers/osowgeneral.htm

45.7.3 Single Trip Permit Information

Non-divisible loads which exceed the annual permit restrictions may be moved by the issuance of a single trip permit.

When a single trip permit is issued, the applicant is required to indicate on the permit the origin and destination of the trip and the highways that are to be used. Another permit is needed for local roads. Each Single Trip Permit vehicle is individually analyzed by WisDOT for all structures that it encounters on the designated permit route.

The load distribution is based on single lane distribution. This is used because these permit loads are infrequent and are likely the only heavy loads on the structure during the crossing. The analysis is done at the operating level.

In special cases the dynamic load allowance (or impact for LFR) may be neglected provided that the maximum vehicle speed can be reduced to 5 MPH prior to crossing the bridge. Also, if some cases, the truck may be escorted across the bridge with no other vehicles allowed on the bridge during the crossing. If this is the case, then the live load factor can be reduced from 1.20 to 1.10 as shown in Table 45.3-3. It is recommended that the truck be centered on the bridge if it is being escorted with no other vehicles allowed on the bridge during the crossing.

45.8 Load Rating Documentation

45.8.1 Load Rating Summary Sheet

After the structure has been load rated, the WisDOT Bridge Load Rating Summary Form shall be completed and utilized as a cover sheet for the load rating calculations (see Figure 45.8-1). This form may be obtained from the Bureau of Structures or is available on the following website:

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

Note: The Load Rating Summary Form is not required to be completed and sent in for concrete box culvert structures.

Instructions for completing the form are as follows:

1. Check what method was used to rate the bridge in the space provided.

2. Enter all data for all items corresponding to the vehicle type. Capacities for the posting vehicles do not have to be calculated if the Operating Rating factor is greater than 1.0 for the HL-93 (LRFR) or the HS20 (LFR or ASR).

3. The rating for the Wis-SPV is always required and should be given on the rating sheet for both a multi-lane distribution and a single-lane distribution. Make sure not to include the future wearing surface in these calculations. All reported ratings are based on current conditions and do not reflect future wearing surfaces.

4. For the Operating Rating, enter the lowest rating for each appropriate vehicle type, subject to the above requirements.

5. For the controlling element, make sure to enter the element (slab, deck girder, lower truss chord, etc.) as well as the check (moment, shear, etc).

6. Be specific in describing where the controlling rating is located. For example, for girder bridges, enter the controlling span, girder-line, and location within the span (Ex. Span 2, G3, midspan).

7. For the live load distribution factor, enter the distribution factor for the controlling element. Be sure to specify if it is a shear distribution factor or a moment distribution factor.

8. Enter all additional remarks as required to clarify the load capacity calculations and, if necessary, recommend posting signage.

9. It is necessary for the responsible engineer to sign and seal the form in the space provided for projects where the Ratings have changed. However, for rehabilitation projects with no change to the Ratings, the Load Rating Summary Form does not need to be signed and sealed.

45.8.2 Load Rating on Plans

The plans shall contain the following rating information:

- Inventory Load Rating – The plans shall have either the HS value of the inventory rating if using LFR or the rating factor for the HL-93 if using LRFR. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. For new concrete box culvert structures, place a rating factor of 1.05 on the plans. See Section 6.2.2.3.4 for more information.

- Operating Load Rating – The plans shall have either the HS value of the operating rating if using LFR or the rating factor for the HL-93 if using LRFR. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. For new concrete box culvert structures, place a rating factor of 1.35 on the plans. See Section 6.2.2.3.4 for more information.

- Wis-SPV – The plans shall also contain the results of the Wis-SPV analysis utilizing single-lane (single trip permit) distribution and assuming that the vehicle is mixing with normal traffic and that the full dynamic load allowance is utilized. This rating shall be based on the current conditions of the bridge at the point when the construction is complete and shall not use the future wearing surface. The recorded rating for this is the total allowable vehicle weight rounded down to the nearest 10 kips. If the value exceeds 250 kips, limit the plan value to 250 kips. For new concrete box culvert structures, place a value of 255 kips for the allowable vehicle weight on the plans. See Section 6.2.2.3.4 for more information.

Note: The culvert ratings indicated above will be used by BOS as a placeholder until policy (AASHTO and WisDOT) is determined for rating culverts.

WISCONSIN DEPARTMENT OF TRANSPORTATION
BRIDGE LOAD RATING SUMMARY

EXISTING BRIDGE DATA (WHEN APPLICABLE)

Bridge Number:	**Last Rating Date:**
Span Type:	**Last Inspection Date:**
Span Material:	**Inspector:**
Design Loading:	**Overburden Depth:**
Inventory Rating:	**Section Loss (SL):**
Operating Rating:	**SL Description:**

BRIDGE LOAD RATING SUMMARY

Rating Method: ☐ LRFR ☐ LFR ☐ ASR (Timber only)

Rating Vehicle: ☐ HL-93 ☐ HS20 ☐ User Defined (Describe in Remarks)

Rating Factors:	**Controlling Element**	**Controlling Location**	**Live Load Distribution Factor**
Inventory			
Operating			

Wisconsin Standard Permit Vehicle (Wis-SPV)

	MVW (kips)	**Controlling Element**	**Controlling Location**	**Live Load Distribution Factor**
Single Lane (w/o FWS)				
Multi Lane (w/o FWS)				

Posting Vehicle Type *	**Vehicle GVW (Kips)**	**Operating Rating (Kips)**	**Controlling Element / Controlling Check**	**Controlling Location**	**Live Load Distribution Factor**
Type 3	50				
Type 3S2	72				
Type 3-3	80				
SU4	54				
SU5	62				
SU6	69.5				
SU7	77.5				
Intermediary Lumber Pup	98				
Intermediary Lumber Semi	98				

* Posting Vehicle Analysis (Run only when HL-93 Operating RF <1.0 or when HS20 Operating RF <1.0)

Remarks / Recommendations:	**Load Rating Engineer**
	Name:
	Date:
	PE Stamp Here

Figure 45.8-1

Bridge Load Rating Summary Form

45.9 Standard Permit Vehicle Moments

Live Load Moments in Foot-Kips per Wheel Line without Impact					
	Span Tenth Points				
Span (Feet)	0.1	0.2	0.3	0.4	0.5
24	71.5	118.0	155.6	178.3	182.4
28	88.6	149.5	191.6	223.0	228.0
32	104.7	180.5	230.8	260.2	273.6
36	121.2	208.6	271.5	310.3	319.2
40	137.0	240.9	311.8	354.1	364.8
44	154.3	271.0	352.8	398.7	409.1
48	173.1	299.1	390.6	446.2	454.6
52	191.0	333.9	430.1	484.3	498.8
56	202.4	359.3	468.2	534.8	547.1
60	241.3	403.5	507.4	573.7	589.5
64	267.7	455.2	562.6	616.8	636.7
68	301.2	506.5	630.3	678.6	683.6
72	329.3	561.1	697.9	746.3	734.6
76	360.1	611.4	765.0	825.7	796.9
80	390.0	667.2	839.0	909.0	869.5
84	426.5	729.2	916.3	994.3	948.4
88	454.9	778.5	985.5	1071.2	1034.5
92	482.5	834.1	1053.6	1155.2	1117.6
96	517.8	889.3	1119.6	1238.8	1201.7
100	543.9	941.4	1197.7	1321.8	1285.2
104	578.5	996.1	1268.4	1404.6	1375.2
108	603.4	1053.4	1342.2	1479.5	1460.8
112	637.3	1099.1	1415.7	1556.5	1543.5
116	662.7	1159.5	1484.0	1640.2	1625.8
120	694.2	1209.0	1549.5	1721.1	1707.9
124	727.5	1257.6	1614.8	1801.7	1789.6
128	750.0	1316.6	1679.8	1882.0	1871.0
132	782.0	1371.2	1745.7	1962.0	1952.1
136	814.5	1411.0	1822.8	2041.8	2037.4

Table 45.9-1

190 KIP Standard Permit Vehicle Live Load Moments on Longitudinal Girders of One Span

Live Load Moments in Foot-Kips per Wheel Line without Impact						
	Span Tenth Points					
Span (Feet)	0.3	0.4	0.5	0.6	0.7	1.0
24	131.3	141.0	139.0	128.2	96.4	-104.4
28	162.5	181.2	175.8	153.5	115.5	-146.7
32	197.6	213.7	210.6	184.7	141.8	-207.7
36	235.2	255.2	250.2	221.4	168.8	-260.5
40	268.5	288.5	284.8	254.8	196.1	-315.8
44	306.2	329.6	319.9	284.3	223.6	-364.9
48	343.0	372.1	359.7	314.0	249.4	-408.1
52	377.7	405.9	394.6	351.4	273.6	-445.9
56	416.8	452.1	435.3	386.9	301.5	-479.8
60	452.3	485.3	472.0	416.6	333.5	-510.8
64	485.2	528.6	512.7	454.5	369.5	-537.6
68	545.0	573.8	553.5	506.3	410.8	-561.0
72	599.0	629.3	594.3	553.7	452.5	-584.4
76	663.1	694.2	643.8	613.3	494.8	-602.2
80	731.0	761.9	701.9	670.9	539.8	-623.2
84	800.5	831.9	768.7	738.2	589.5	-639.5
88	864.1	903.3	835.8	802.8	633.2	-655.4
92	926.3	974.1	903.2	865.1	680.4	-670.2
96	988.9	1046.9	980.8	928.1	722.9	-680.2
100	1061.2	1120.6	1059.7	991.5	777.4	-713.5
104	1129.3	1195.7	1138.9	1055.5	825.1	-757.6
108	1202.0	1263.7	1210.4	1118.0	877.4	-799.3
112	1275.3	1334.1	1279.6	1189.6	930.5	-842.1
116	1343.7	1411.6	1349.2	1256.4	979.1	-885.6
120	1410.1	1487.4	1422.1	1321.0	1025.7	-926.9
124	1476.8	1563.6	1500.3	1385.9	1072.7	-967.2
128	1543.7	1640.4	1578.9	1451.0	1120.2	-1008.7
132	1611.1	1717.9	1657.6	1516.7	1168.5	-1050.9
136	1678.8	1795.7	1736.6	1582.7	1224.2	-1092.2

Table 45.9-2

190 KIP Standard Permit Vehicle Live Load Moments on Longitudinal Girders of Two Equal Length Spans Constant Moment of Inertia

45.10 References

1. *Final Report on Full-Scale Bridge Testing* by E. G. Burdette and D. W. Goodpasture, Department of Civil Engineer, University of Tennessee, 1971.

2. *The AASHTO Road Test*, Report 4 Bridge Research by the Highway Research Board, Washington, D.C. 1962.

3. *Standard Specifications for Highway Bridges* by American Association of State Highway and Transportation Officials.

4. *AASHTO LRFD Bridge Design Specifications* by American Association of State Highway and Transportation Officials, 2012.

5. *The Manual for Bridge Evaluation* by American Association of State Highway and Transportation Officials, 2011.

6. *Structure Inspection Manual* by Wisconsin Department of Transportation, 2003.

7. *Reinforced Concrete Design* by C. K. Wang and C. G. Salmon.

8. *Plastic Design of Steel Frames* by Lynn S. Beedle.

9. National Cooperative Highway Research Program Report 312.

10. National Cooperative Highway Research Program Project 12-63.

11. *Post-Tensioning Manual* by Post-Tensioning Institute.

12. Wisconsin Statutes, Vol. 4, Chapter 348.

13. *Summary of Motor Vehicle Size and Weight Regulations in Wisconsin* by Dept. of Transportation, Division of Motor Vehicles.

45.11 Rating Examples

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):

$$a := \frac{A_s\cdot f_s}{0.85\cdot f'_c\cdot b}$$

where:

A_s = area of developed reinforcement at section (in^2)

f_s = stress in reinforcement (ksi)

$f'_c = 4$ ksi

$b := 12$ in

As shown throughout the Chapter 18 Example, when f_s is assumed to be equal to f_y, and is used to calculate (a), the value of c/d_s will be < 0.6 (for f_y = 60 ksi) per **LRFD [5.7.2.1]** Therefore the assumption that the reinforcement will yield ($f_s = f_y$) is correct. The value for (c) and (d_s) are calculated as:

$$c := \frac{a}{\beta_1}$$

$\beta_1 := 0.85$

d_s = slab depth(excl. 1/2" wearing surface) - bar clearance - 1/2 bar diameter

For rectangular sections, the nominal moment resistance, M_n, (tension reinforcement only) equals:

$$M_n := A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

Minimum Reinforcement Check

All sections throughout the bridge meet minimum reinforcement requirements, because this was checked in the chapter 18 Design example. Therefore, no adjustment to nominal resistance (M_n) or moment capacity is required. **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

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:

$$RF := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88$ $\frac{in^2}{ft}$

$d_s := 28.0 - 2.0 - 0.5$ $\boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{0.85\cdot f'_c\cdot b}$ $\boxed{a = 2.76}$ 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:

$RF_{permit}\cdot(190) = 310 \quad$ 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:

$$RF := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC}) - (\gamma_{DW})\cdot(M_{DW})}{\gamma_L\cdot(M_{LL\_IM})}$$

$A_{st\_pier} := 1.88 \quad \frac{in^2}{ft}$

$d_s := 28.0 - 2.0 - 0.5 \qquad \boxed{d_s = 25.5}$ in

$a := \frac{A_{st\_pier}\cdot f_y}{0.85\cdot f'_c\cdot b} \qquad \boxed{a = 2.76}$ 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

$M_{DC} := 59.2\,\text{kip} - \text{ft}$ (from Chapter 18 Example, Table E18.4)

The live load moment at the C/L of Pier due to the Wisconsin Standard Permit Vehicle (Wis-SPV) having a gross vehicle load of 190 kips and utilizing single lane distribution is:

$M_{LL\_IM} := 65.2$ kip – ft

Permit:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi) \cdot M_n - (\gamma_{DC}) \cdot (M_{DC})}{\gamma_L \cdot (M_{LL\_IM})}$$

$\boxed{RF_{permit} = 1.66}$

The maximum Wisconsin Standard Permit Vehicle (Wis_SPV) load is:

$RF_{permit} \cdot (190) = 316$ kips

This same procedure used for the (Wis-SPV) can also be used when evaluating the bridge for an actual "Single-Trip Permit" vehicle.

E45-1.2.6.3 Wis-SPV Permit Rating with Multi Lane Distribution w/o FWS

Rating for Flexure

$$RF := \frac{(\phi_c)(\phi_s)(\phi) \cdot M_n - (\gamma_{DC}) \cdot (M_{DC})}{\gamma_L \cdot (M_{LL\_IM})}$$

The capacity of the bridge to carry the Permit Vehicle Load was checked at 0.1 pts. along the structure and at the slab/haunch intercepts. The governing location is at the C/L of Pier.

Load Factors

$\gamma_{DC} := 1.25$	Chapter 45 Table 45.3-1
$\gamma_{DW} := 1.50$	WisDOT policy is to always use 1.50; Chapter 45 Table 45.3-1
$\gamma_L := 1.30$	WisDOT Policy when analyzing the Wis-SPV as an "Annual Permit" vehicle with no escorts

At C/L of Pier

Permit Vehicle:

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$M_n = 226.7$ kip – ft (as shown previously)

$M_{DC} = 59.2$ kip – ft (as shown previously)

The live load moment at the C/L of Pier due to the Wisconsin Permit Vehicle (Wis_SPV) having a gross vehicle load of 190 kips and a DF of 0.0851 lanes/ft-slab:

$M_{LL\_IM} := 98.7$ kip – ft

$$RF_{permit} := \frac{(\phi_c)(\phi_s)(\phi)\cdot M_n - (\gamma_{DC})\cdot(M_{DC})}{\gamma_L\cdot(M_{LL\_IM})}$$

$$\boxed{RF_{permit} = 1.01}$$

The Wisconsin Standard Permit Vehicle (Wis_SPV) load that can be carried by the bridge is:

$RF_{permit}\cdot(190) = 193$ kips

E45-1.3 Summary of Rating

Slab - Interior Strip							
Limit State		Design Load Rating		Legal Load Rating	Permit Load Rating (kips)		
		Inventory	Operating		Single DF w/ FWS	Single DF w/o FWS	Multi DF w/o FWS
Strength I	Flexure	1.04	1.34	N/A	310	316	190
Service I		N/A	N/A	N/A	Optional	Optional	Optional

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

$$RF_{Legal} := \frac{(1)(1)(1)(M_n) - \gamma_{DC}\cdot(M_{DC1} + M_{DC2})}{\gamma_{L\_Legal}\cdot(M_{LLIM})}$$

$$RF_{SU} := \frac{(1)(1)(1)(M_n) - \gamma_{DC}\cdot(M_{DC1} + M_{DC2})}{\gamma_{L\_SU}\cdot(M_{LLIM})}$$

AASHTO Type	Truck Type	Truck Weight (Tons)	M_{LL} (Per Lane) (ft-kips)	M_{LLIM} (M_{LL} * IM * g_i) ft-kips	RF Strength I Flexure	Safe Load Capacity (Tons)	Posting?
Commercial Trucks	Type 3	25	1671.0	1413.4	4.520	113	No
	Type 3S2	36	2150.0	1818.6	3.513	126	No
	Type 3-3	40	2260.0	1911.7	3.342	134	No
Specialized Hauling Vehicles	SU4	27	1831.0	1548.8	4.124	111	No
	SU5	31	2062.8	1744.9	3.661	113	No
	SU6	34.75	2294.6	1940.9	3.291	114	No
	SU7	38.75	2540.8	2149.2	2.972	115	No

As expected, all rating factors are well above 1.0. However, if any of the rating factors would have fallen below 1.0, the posting capacity would have been calculated per 45.3.2.7.2:

$$\text{Posting} := \left(\frac{W}{0.7}\right)[(RF) - 0.3]$$

E45-2.11 Permit Load Rating

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per 45.6.

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 allowance is utilized. Future wearing surface shall be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW.

Also, divide out the 1.2 multiple presence factor per **LRFR [6.4.5.4.2.2]** for the single lane distribution factor run.

For 146' span:

$M190_{LL} := 4930.88$ kip-ft per lane

$V190_{LL} := 145.08$ kips at $\boxed{d_v = 65}$ in

for Strength Limit State

Single Lane Distribution w/ Future Wearing surface (Design check per 45.6)

$g_{m1} := 0.435 \frac{1}{1.2}$ $\boxed{g_{m1} = 0.363}$

$g_{v1} := .660 \cdot \frac{1}{1.2}$ $\boxed{g_{v1} = 0.550}$

For flexure:

$M_{190LLIM} := M190_{LL} \cdot g_{m1} \cdot 1.33$ $\boxed{M_{190LLIM} = 2377}$ kip-ft

$$RF_{190\_moment} := \frac{\left[(1)(1)(1)M_n\right] - 1.25 \cdot \left(M_{DC1} + M_{DC2}\right) - 1.5 \cdot \left(M_{DW}\right)}{1.2\left(M_{190LLIM}\right)}$$

$\boxed{RF_{190\_moment} = 3.060}$

$Wt := RF_{190\_moment} \cdot 190$ $\boxed{Wt = 581}$ kips >> 190 kips, OK

For shear:

$V_{190LLIM} := V190_{LL} \cdot g_{v1} \cdot 1.33$ $\boxed{V_{190LLIM} = 106}$ kips

$$RF_{190\_shear} := \frac{\left[(1)(1)(0.9)V_n\right] - 1.25 \cdot \left(V_{DCnc} + V_{DCc}\right) - 1.5\left(V_{DW}\right)}{1.2\left(V_{190LLIM}\right)}$$

$\boxed{RF_{190\_shear} = 1.418}$

$Wt := RF_{190\_shear} \cdot 190$ $\boxed{Wt = 269}$ kips > 190 kips, OK

Single Lane Distribution w/o Future Wearing surface (For plans and rating sheet only)

$g_{m1} := 0.435 \frac{1}{1.2}$ $\boxed{g_{m1} = 0.363}$

$g_{v1} := .660 \cdot \frac{1}{1.2}$ $\boxed{g_{v1} = 0.550}$

For flexure:

$M_{190LLIM} := M190_{LL} \cdot g_{m1} \cdot 1.33$ $\boxed{M_{190LLIM} = 2377}$ kip-ft

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25\cdot(M_{DC1} + M_{DC2})}{1.2(M_{190LLIM})}$$

$$\boxed{RF_{190\_moment} = 3.247}$$

$$Wt := RF_{190\_moment}\cdot 190 \qquad \boxed{Wt = 617}$$

For shear:

$$V_{190LLIM} := V190_{LL}\cdot g_{v1}\cdot 1.33 \qquad \boxed{V_{190LLIM} = 106} \text{ kips}$$

$$RF_{190\_shear} := \frac{[(1)(1)(0.9)V_n] - 1.25\cdot(V_{DCnc} + V_{DCc})}{1.2(V_{190LLIM})}$$

$$\boxed{RF_{190\_shear} = 1.533}$$

$$Wt := RF_{190\_shear}\cdot 190 \qquad \boxed{Wt = 291}$$

Multi-Lane Distribution w/o Future Wearing Surface (For plans and rating sheet only)

$$g_{m2} := 0.636 \qquad \boxed{g_{m2} = 0.636}$$

$$g_{v2} := .779 \qquad \boxed{g_{v2} = 0.779}$$

For flexure:

$$M_{190LLIM} := M190_{LL}\cdot g_{m2}\cdot 1.33 \qquad \boxed{M_{190LLIM} = 4171} \text{ kip-ft}$$

$$RF_{190\_moment} := \frac{[(1)(1)(1)M_n] - 1.25\cdot(M_{DC1} + M_{DC2})}{1.3(M_{190LLIM})}$$

$$\boxed{RF_{190\_moment} = 1.708}$$

$$Wt := RF_{190\_moment}\cdot 190 \qquad \boxed{Wt = 325}$$

For shear:

$V_{190LLIM} := V190_{LL} \cdot g_{v2} \cdot 1.33$ $\boxed{V_{190LLIM} = 150}$ kips

$$RF_{190\_shear} := \frac{(1)(1)(0.9)\,V_n - 1.25 \cdot \left(V_{DCnc} + V_{DCc}\right)}{1.3\left(V_{190LLIM}\right)}$$

$\boxed{RF_{190\_shear} = 0.999}$

$Wt := RF_{190\_shear} \cdot 190$ $\boxed{Wt = 190}$

E45-2.12 Summary of Rating Factors

Interior Girder								
Limit State		Design Load Rating		Legal Load Rating	Permit Load Rating (kips)		
		Inventory	Operating		Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Strength I	Flexure	1.723	2.233	N/A	581	617	325
	Shear	1.11	1.439	N/A	269	291	190
Service III		1.43	N/A	N/A	N/A	N/A	N/A
Service I		N/A	N/A	N/A	Optional	Optional	Optional

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This is approximately equal to the thickness of the bottom flange height of 7.5 inches.

$$M_n := As \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \cdot \frac{1}{12} \qquad \boxed{M_n = 7544} \text{ kip-ft}$$

$$M_r := \phi_f \cdot M_n \qquad \boxed{M_r = 6790} \text{ kip-ft}$$

E45-3.12 Design Load Rating

This design example illustrates the rating checks required at the location of maximum negative moment. These checks are also required at the locations of continuity bar cut offs but are not shown here.

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi) R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L (LL + IM)}$$

Load Factors taken from Table 45.3-1

$\gamma_{L\_inv} := 1.75$ $\quad \gamma_{DC} := 1.25$ $\quad \gamma_{servLL} := 0.8$ $\quad \phi_c := 1.0$ $\quad \phi_s := 1.0$

$\gamma_{L\_op} := 1.35$ $\quad \gamma_{DW} := 1.50$ $\quad \phi := 0.9$ for flexure

For Flexure

$\boxed{M_n = 7544}$ kip-ft $\quad \boxed{M_{DCc} = 272}$ kip-ft $\quad \boxed{M_{LL} = 2055}$ kip-ft

Inventory Level

$$RF_{Mom\_Inv} := \frac{(\phi_c)(\phi_s)(\phi)(M_n) - \gamma_{DC} \cdot (M_{DCc})}{\gamma_{L\_inv} \cdot (M_{LL})} \qquad \boxed{RF_{Mom\_Inv} = 1.793}$$

Operating Level

$$RF_{Mom\_Op} := \frac{(\phi_c)(\phi_s)(\phi)(M_n) - \gamma_{DC} \cdot (M_{DCc})}{\gamma_{L\_op} \cdot (M_{LL})} \qquad \boxed{RF_{Mom\_Op} = 2.325}$$

E45-3.13 Permit Load Rating

Check the Wisconsin Standard Permit Vehicle per 45.6

For a symetric 130' two span structure:

$MSPV_{LL} := 2738$ kip-ft per lane (includes Dynamic Load Allowance of 33%)

Per 45.6, for the Wisconsin Standard Permit Vehicle (Wis-SPV) Design Check use single lane distribution factor assuming a single trip permit vehicle with no escort vehicles and assuming full dynamic load allowance. Also, divide out the 1.2 multiple presence factor per **LRFR [6.4.5.4.2.2]** for the single lane distribution factor only.

Single Lane Distribution

$$g_1 := g_{i1}\frac{1}{1.2} \qquad \boxed{g_1 = 0.356}$$

$$M_{SPVLLIM} := \left(MSPV_{LL} + M_{Lane}\right)\cdot g_1 \qquad \boxed{M_{SPVLLIM} = 975} \text{ kip-ft}$$

$$RF_{SPV\_m1} := \frac{\left[\left(\phi_c\right)\left(\phi_s\right)\left(\phi\right)\left(M_n\right)\right] - 1.25\cdot\left(M_{DCc}\right) - 1.5\left(M_{DWc}\right)}{1.2\left(M_{SPVLLIM}\right)} \qquad \boxed{RF_{SPV\_m1} = 5.151}$$

$$Wt_1 := RF_{SPV\_m1}\cdot 190 \qquad \boxed{Wt_1 = 979} \text{ kips >> 190 kips, OK}$$

The rating for the Wis-SPV vehicle is now checked without the Future Wearing Surface. This value is reported on the plans.

$$RF_{SPV\_m\_pln} := \frac{\left[\left(\phi_c\right)\left(\phi_s\right)\left(\phi\right)\left(M_n\right)\right] - 1.25\cdot\left(M_{DCc}\right)}{1.2\left(M_{SPVLLIM}\right)} \qquad \boxed{RF_{SPV\_m\_pln} = 5.511}$$

$$Wt_{pln} := RF_{SPV\_m\_pln}\cdot 190 \qquad \boxed{Wt_{pln} = 1047} \text{ kips}$$

Since this value is greater than 250 kips, 250 kips is reported on the plans and on the Bridge Load Rating Summary form for the single-lane Permit Load Rating.

Multi-Lane Distribution

$$g_2 := g_{i2} \qquad \boxed{g_2 = 0.619}$$

$$M_{SPVLLIM} := MSPV_{LL}\cdot g_2 \qquad \boxed{M_{SPVLLIM} = 1696} \text{ kip-ft}$$

$$RF_{SPV\_m2} := \frac{\left[(\phi_c)(\phi_s)(\phi)(M_n)\right] - 1.25\cdot(M_{DCc})}{1.3(M_{SPVLLIM})}$$ $\boxed{RF_{SPV\_m2} = 2.925}$

$$Wt_2 := RF_{SPV\_m2}\cdot 190$$ $\boxed{Wt_2 = 556}$ kips

Since this value is greater than 250 kips, 250 kips is reported on the Bridge Load Rating Summary form for the multi-lane Permit Load Rating.

E45-3.14 Summary of Rating Factors

Interior Girder						
Limit State		Design Load Rating		Legal Load	Permit Load Rating (kips)	
		Inventory	Operating	Rating	Single Lane	Multi-Lane
Strength 1	Flexure	1.79	2.32	N/A	250	250

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$V_{DC2} = -12.03$ kips

$V_{LL} = -131.95$ kips

$M_{LLIM\_neg} = -2414.17$ ft – kips

E45-4.15 Design Load Rating @ Pier for Shear

$$RF := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC}(V_{DC})}{\gamma_L(V_{LLIM})}$$

Where:

Load Factors per Table 45.3-1

$\gamma_{Linv} := 1.75$

$\gamma_{Lop} := 1.35$

$\gamma_{DC} := 1.25$

Resistance Factors

$\phi := 1.0$ **LRFR [6.7.3]**

$\phi_c := 1.0$ per 45.3.2.4

$\phi_s := 1.0$ per 45.3.2.5

A. Strength Limit State

Inventory

$$RF_{inv\_shear} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-V_n) - \gamma_{DC} \cdot (V_{DC1} + V_{DC2})}{\gamma_{Linv} \cdot (V_{LL})}$$

$\boxed{RF_{inv\_shear} = 1.58}$

Operating

$$RF_{op\_shear} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot (-V_n) - \gamma_{DC} \cdot (V_{DC1} + V_{DC2})}{\gamma_{Lop} \cdot (V_{LL})}$$

$\boxed{RF_{op\_shear} = 2.05}$

Since RF>1.0 @ operating for all checks, Legal Load Ratings are not required for this example.

E45-4.16 - Permit Load Ratings

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed (per 45.6). Since the span lengths are less than 200', the lane loading requirements will not be considered for positive moments.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic load allowance is utilized. Future wearing surface shall not be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW. Future wearing surface shall be included in the check.

E45-4.16.1 - Wis-SPV Permit Rating with Single Lane Distribution w/ FWS

The values from this analysis are used for performing the Wis-SPV design check per 45.6

Load Distribution Factors

Single Lane Interior DF - Moment $g_{m1} = 0.47$

Single Lane Interior DF - Shear $g_{v1} = 0.75$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.2$

$\gamma_{DC} := 1.25$ $\gamma_{DW} := 1.50$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

$M_{0.4L} := \frac{g_{m1}}{1.2} \cdot 1.33 \cdot M_{pos}$ $\boxed{M_{0.4L} = 1468.47}$ kip-ft

$M_{1.0L} := \left(\frac{g_{m1}}{1.2}\right) \cdot \left(\left(1.33 \cdot M_{neg}\right)\right)$ $\boxed{M_{1.0L} = 1129.31}$ kip-ft

$$V_{1.0L} := \left(\frac{g_{v1}}{1.2}\right)\cdot((1.33\cdot V_{max}))$$ $V_{1.0L} = 128.28$ kips

$$RF_{pos} := \frac{\phi\cdot\phi_c\cdot\phi_s\cdot M_{n\_0.4L} - \gamma_{DC}\cdot(M_{DC1} + M_{DC2}) - \gamma_{DW}\cdot M_{DW}}{\gamma_L\cdot(M_{0.4L})}$$

$\boxed{RF_{pos} = 2.55}$ $\boxed{RF_{pos}\cdot 190 = 483.65}$ kips

$$RF_{neg} := \frac{\phi\cdot\phi_c\cdot\phi_s\cdot M_{n\_1.0L} - \gamma_{DC}\cdot(-M_{DC1\_neg} - M_{DC2\_neg}) - \gamma_{DW}\cdot(-M_{DW\_neg})}{\gamma_L\cdot(M_{1.0L})}$$

$\boxed{RF_{neg} = 3.74}$ $\boxed{RF_{neg}\cdot 190 = 711.43}$ kips

$$RF_{shear} := \frac{\phi\cdot\phi_c\cdot\phi_s\cdot V_n - \gamma_{DC}\cdot[-(V_{DC1} + V_{DC2})] - \gamma_{DW}\cdot(-V_{DW})}{\gamma_L\cdot(V_{1.0L})}$$

$\boxed{RF_{shear} = 2.24}$ $\boxed{RF_{shear}\cdot 190 = 424.87}$ kips

424.87k > 190k minimum : CHECK OK

E45-4.16.2 - Wis-SPV Permit Rating with Single Lane Distribution w/o FWS

For use with plans and rating sheet only.

Load Distribution Factors

Single Lane Interior DF - Moment $g_{m1} = 0.47$

Single Lane Interior DF - Shear $g_{v1} = 0.75$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.2$

$\gamma_{DC} := 1.25$ $\gamma_{DW} := 1.50$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance
or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

$$M_{0.4L} := \frac{g_{m1}}{1.2} \cdot 1.33 \cdot M_{pos}$$ $\boxed{M_{0.4L} = 1468.47}$ kip-ft

$$M_{1.0L} := \left(\frac{g_{m1}}{1.2}\right) \cdot \left(\left(1.33 \cdot M_{neg}\right)\right)$$ $\boxed{M_{1.0L} = 1129.31}$ kip-ft

$$V_{1.0L} := \left(\frac{g_{v1}}{1.2}\right) \cdot \left(\left(1.33 \cdot V_{max}\right)\right)$$ $V_{1.0L} = 128.28$ kips

$$RF_{pos1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot \left(M_{DC1} + M_{DC2}\right)}{\gamma_L \cdot \left(M_{0.4L}\right)}$$

$\boxed{RF_{pos1} = 2.68}$ $\boxed{RF_{pos1} \cdot 190 = 508.78}$ kips

$$RF_{neg1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC} \cdot \left(-M_{DC1\_neg} - M_{DC2\_neg}\right)}{\gamma_L \cdot \left(M_{1.0L}\right)}$$

$\boxed{RF_{neg1} = 4.16}$ $\boxed{RF_{neg1} \cdot 190 = 789.64}$ kips

$$RF_{shear1} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC} \cdot \left[-\left(V_{DC1} + V_{DC2}\right)\right]}{\gamma_L \cdot \left(V_{1.0L}\right)}$$

$\boxed{RF_{shear1} = 2.37}$ $\boxed{RF_{shear1} \cdot 190 = 450.24}$ kips

E45-4.16.3 - Permit Rating with Multi-Lane Distribution w/o FWS

For use with plans and rating sheet only.

Load Distribution Factors

Multi Lane Interior DF - Moment $g_{m2} = 0.69$

Multi Lane Interior DF - Shear $g_{v2} = 0.93$

Load Factors per Tables 45.3-1 and 45.3-3

$\gamma_L := 1.3$

$\gamma_{DC} := 1.25$

Wis-SPV Moments and Shears (w/o Dynamic Load allowance
or Distribution Factors included)

$M_{pos} := 2842.10$ kip-ft

$M_{neg} := 2185.68$ kip-ft

$V_{max} := 154.32$ kips

Multi Lane Ratings

$M_{0.4L} := g_{m2} \cdot 1.33 \cdot M_{pos}$ $\boxed{M_{0.4L} = 2600.09}$ kip-ft

$M_{1.0L} := g_{m2} \cdot (1.33 \cdot M_{neg})$ $\boxed{M_{1.0L} = 1999.56}$ kip-ft

$V_{1.0L} := g_{v2} \cdot (1.33 \cdot V_{max})$ $\boxed{V_{1.0L} = 191.88}$ kips

$$RF_{pos\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_0.4L} - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_L \cdot (M_{0.4L})}$$

$\boxed{RF_{pos\_ml} = 1.40}$ $\boxed{RF_{pos\_ml} \cdot 190 = 265.24}$ kips

$$RF_{neg\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot M_{n\_1.0L} - \gamma_{DC} \cdot \left(-M_{DC1\_neg} - M_{DC2\_neg}\right)}{\gamma_L \cdot \left(M_{1.0L}\right)}$$

$\boxed{RF_{neg\_ml} = 2.17}$ $\boxed{RF_{neg\_ml} \cdot 190 = 411.67}$ kips

$$RF_{shear\_ml} := \frac{\phi \cdot \phi_c \cdot \phi_s \cdot V_n - \gamma_{DC} \cdot \left[-\left(V_{DC1} + V_{DC2}\right)\right]}{\gamma_L \cdot \left(V_{1.0L}\right)}$$

$\boxed{RF_{shear\_ml} = 1.46}$ $\boxed{RF_{shear\_ml} \cdot 190 = 277.84}$ kips

E45-4.17 Summary of Rating

Steel Interior Girder							
Limit State		Design Load Rating		Legal Load Rating	Wis-SPV Ratings (kips)		
		Inventory	Operating		Single Lane w/ FWS	Single Lane w/o FWS	Multi Lane w/o FWS
Strength I @ 0.4L	Flexure	1.41	1.82	N/A	484	509	265
	Shear	N/A	N/A	N/A	N/A	N/A	N/A
Strength I @ 1.0L	Flexure	1.30	1.68	N/A	711	790	412
	Shear	1.58	2.05	N/A	425	450	278
Service II	0.4L	1.38	1.80	N/A	Optional		Optional
	1.0L	1.88	2.44	N/A	Optional		Optional