Table of Contents

2.1 Organizational Charts

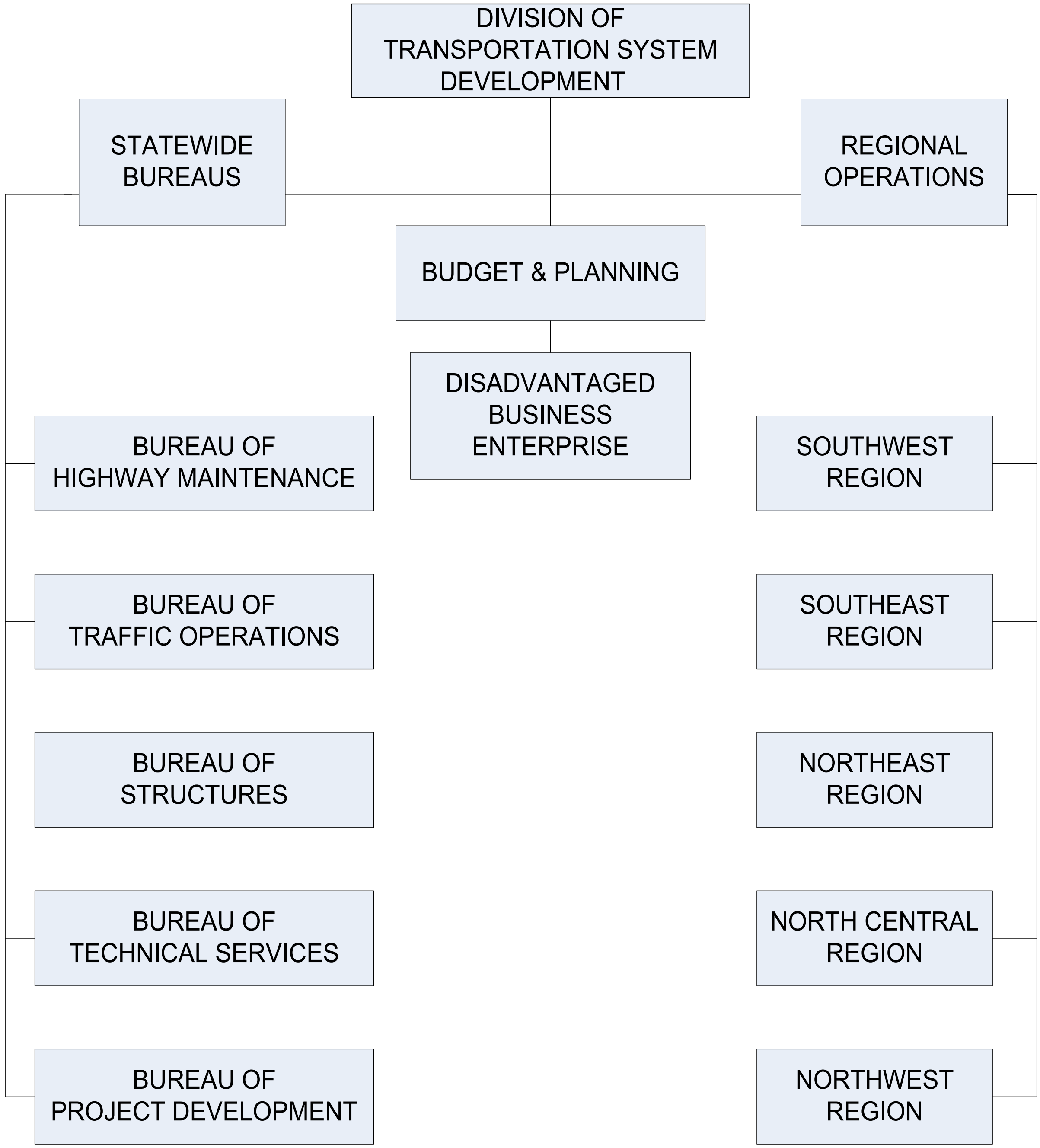


Figure 2.1-1
Division of Transportation System Development

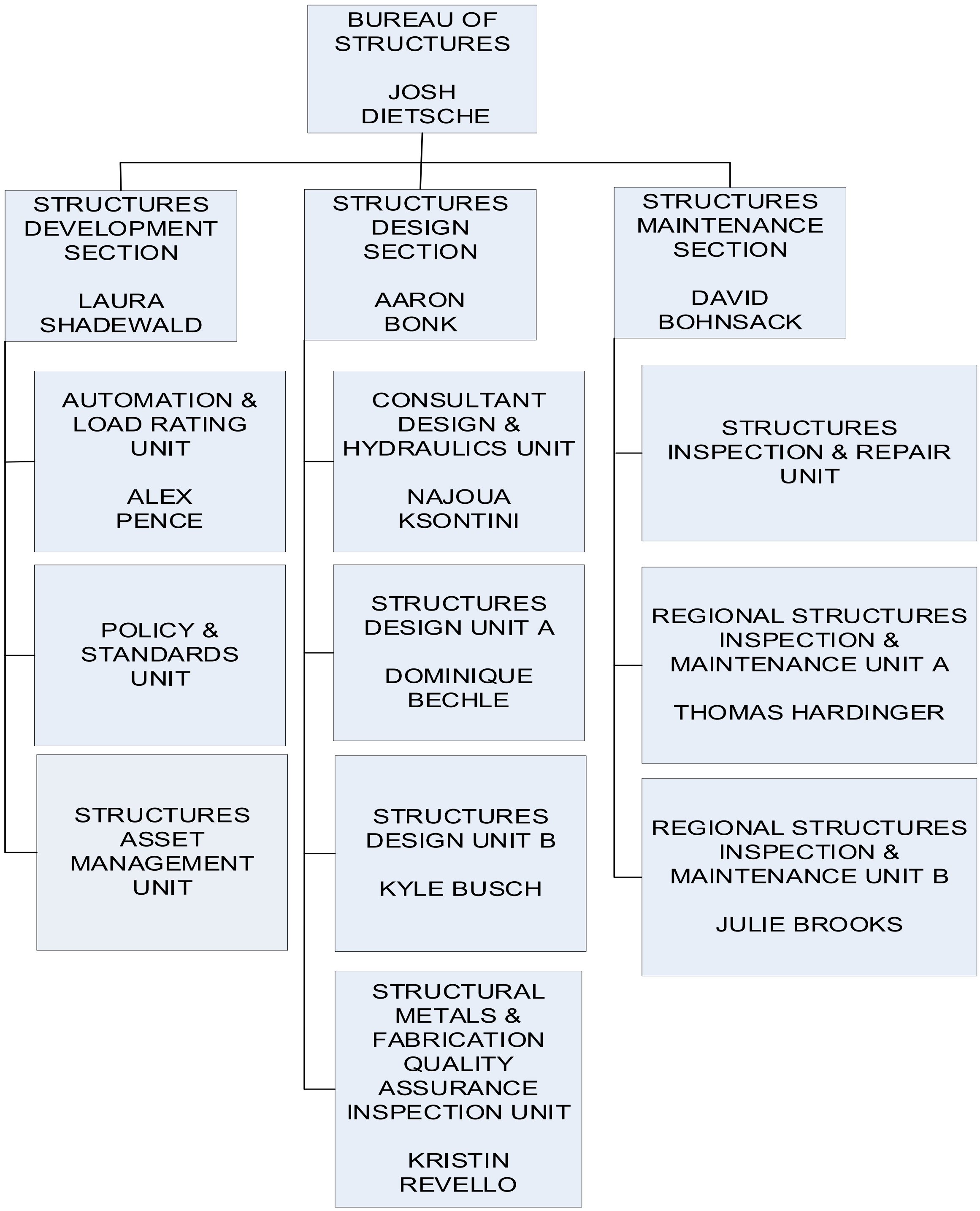
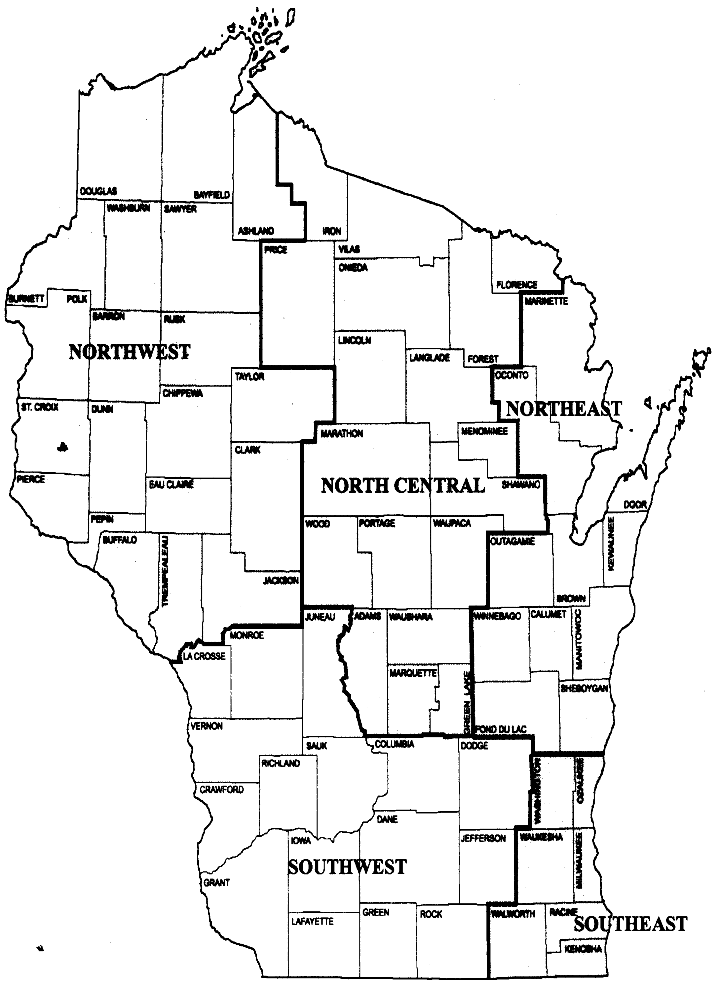


Figure 2.1-2
Bureau of Structures



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

Walls having an exposed height of less than 5.5 ft. measured from the plan ground line to top of wall may require an R number based on specific project features. Designer to contact the Bureau of Structures region liaison for more information.

- N is assigned to noise barriers. Unit numbers may be assigned to long bridges or complex interchanges where it is desirable to have only one structure number for the site.

- M is assigned to miscellaneous structures where it is desirable to have a structure plan record while not meeting the above-mentioned structure assigned criteria.

Numeric code for the County where the bridge is located.
Leading zeros are not shown.

B-5-70 B-40-702-2

Unique Bridge Number assigned for that County by the Regional Office.

Unit Number (if used)

Figure 2.5-1
Bridge Number Detail

2.6 Bridge Files

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

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

Table of Contents

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

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

6.5.1 Approvals, Distribution, and Work Flow

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

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

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Preliminary Plan Requirements

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

1. Hydrology Report

2. Structure Survey Report

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

4. Evaluation of Site Investigation Report

5. Contour map

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

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

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

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

6.5.3 Final Plan Requirements

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

1. Final Drawings

2. Design and Quantity Computations

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

3. Final Site Investigation Report

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

5. QA/QC Verification Sheet

6. Inventory Data Sheet

7. Bridge Load Rating Summary Form

8. LRFD Input File (Excel ratings spreadsheet)

9. On-Time Improvement Form

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

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

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

6.5.4 Addenda

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

6.5.5 Post-Let Revisions

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

6.5.6 Local-Let Projects

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

6.5.7 Locally-Funded Projects

Local highway bridges designed under authority of the local municipality or county highway department without utilizing state or federal oversight must provide to BOS at minimum the following documents: (1) Bridge Inventory Data Sheet, (2) Scour assessment/evaluation documentation, (3) Bridge load rating summary sheet, and (4) Construction documentation such as an as-built plan and shop drawings. The scour assessment/evaluation documentation should be prepared in accordance with 8.3.2.7. All bridges shall be evaluated to determine the vulnerability to scour. See Chapter 8 – Hydraulics for additional guidance on hydrologic and hydraulic analysis.

6.6 Structures Data Management and Resources

6.6.1 Structures Data Management

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Table 6.6-1
Various Inspection Reports

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

6.6.2 Resources

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

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

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

Table of Contents

28" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	59	65
6’-6”	58	63
7’-0”	56	62
7’-6”	55	60
8’-0”	54	59
8’-6”	52	57
9’-0”	51	56
9’-6”	50	54
10’-0”	49	53
10’-6”	48	52
11’-0”	47	51
11’-6”	46	50
12’-0”	45	48

36W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	94	101
6’-6”	92	99
7’-0”	90	97
7’-6”	88	95
8’-0”	87	93
8’-6”	85	91
9’-0”	83	90
9’-6”	82	87
10’-0”	80	86
10’-6”	79	84
11’-0”	77	82
11’-6”	76	81
12’-0”	73	79

45W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	111	120
6’-6”	109	117
7’-0”	107	115
7’-6”	105	113
8’-0”	103	111
8’-6”	101	108
9’-0”	99	106
9’-6”	97	104
10’-0”	95	102
10’-6”	94	100
11’-0”	92	98
11’-6”	90	96
12’-0”	88	94

54W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	125	134
6’-6”	123	132
7’-0”	120	129
7’-6”	118	127
8’-0”	116	125
8’-6”	114	122
9’-0”	112	120
9’-6”	110	117
10’-0”	108	115
10’-6”	106	114
11’-0”	104	111
11’-6”	103	110
12’-0”	100	107

Table 19.3-1
Maximum Span Length vs. Girder Spacing

72W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	153*	164*⊗
6'-6"	150	161*⊗
7'-0"	148	158*
7'-6"	145	156*
8'-0"	143	153*
8'-6"	140	150
9'-0"	138	148
9'-6"	135	144
10'-0"	133	142
10'-6"	131	140
11'-0"	129	137
11'-6"	127	135
12'-0"	124	132

82W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	166*⊗	177*⊗
6'-6"	163*⊗	174*⊗
7'-0"	161*⊗	172*⊗
7'-6"	158*	169*⊗
8'-0"	156*	166*⊗
8'-6"	152	163*⊗
9'-0"	150	160*⊗
9'-6"	147	157*
10'-0"	145	154*
10'-6"	143	152
11'-0"	140	149
11'-6"	138	147
12'-0"	135	144

Table 19.3-2
Maximum Span Length vs. Girder Spacing

* Span length requires a lifting check with pick-up points at the 1/10 points from the end of the girder and a note should be placed on the girder details sheet to reflect that the girder was analyzed for a potential lift at the 1/10 point. For lateral stability during lifting these girder lengths may require pick-up point locations greater than distance d (girder depth) from the ends of the girder, as stated in the Standard Specifications. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange, if required, and check the concrete strength near the lift location based on f'_{ci}.

⊗ Due to difficulty manufacturing, transporting and erecting excessively long prestressed girders, consideration should be given to utilizing an extra pier to minimize use of such girders. Approval from the Bureau of Structures is required to utilize any girder over 158 ft. long. (Currently, there is still a moratorium on the use of all 82W"). Steel girders may be considered if the number of piers can be reduced enough to offset the higher costs associated with a steel superstructure.

Table of Contents

E19-4 Lifting Check for Prestressed Girders, LRFD

This example shows design calculations for the lifting check for the girder in design example E19-1. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2016 Interim)*** NOTE: A lifting check at the 1/10th point is only required for long spans, as discussed in Table 19.3-2 notes. Since this example is not considered a long span, the following lifting check at the 1/10th point is not required and should be used for informational purposes only.

E19-4.1 Design Criteria

$L_g := 147$ feet

$f'_{ci} := 6.8$ ksi $\qquad f_y := 60$ ksi

$girder_size = $ "72W-inch"

$w_{top\_flg} = 48$ inches $\qquad w_{girder} = 0.953$ kips/ft

$t_{top\_flg\_min} = 3.0$ inches $\qquad S_{bot} = -18825$ in^3

$t_{top\_flg\_max} = 5.5$ inches $\qquad S_{top} = 17680$ in^3

$t_w = 6.5$ inches

Lift point is assumed to be at the 1/10th point of the girder length.

E19-4.2 Lifting Stresses

Initial Girder Stresses (Taken from Prestressed Girder Output):

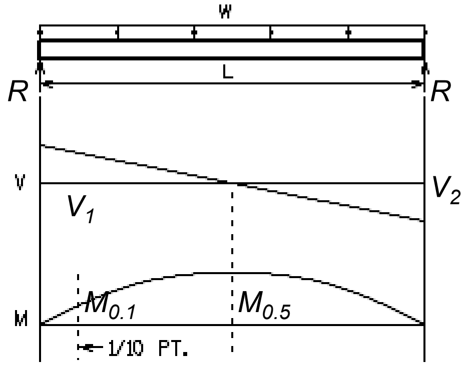
At the 1/10th Point, (positive values indicate compression)

$f_{i\_top\_0.1} := 0.284$ ksi

$f_{i\_bot\_0.1} := 3.479$ ksi

The initial stresses in the girder (listed above) are due to the prestressed strands and girder dead load moment. The girder dead load moment and resulting stresses are based on the girder being simply supported at the girder ends. These resulting stresses are subtracted from the total initial stresses to give the stresses resulting from the pressing force alone.

Moments and Shears due to the girder self weight:



$R := \frac{1}{2} \cdot (w_{girder}) \cdot L_g$ $\quad R = 70.05$ kips

$V_1 := R$ $\quad V_1 = 70.05$ kips

$V_2 := R$ $\quad V_2 = 70.05$ kips

$$M_{gird0.1} := \frac{(w_{girder}) \cdot (0.1 \cdot L_g)}{2} \cdot (L_g - 0.1 \cdot L_g)$$

$M_{gird0.1} = 926.7$ kip-ft

Top of girder stresses due to prestress forces:

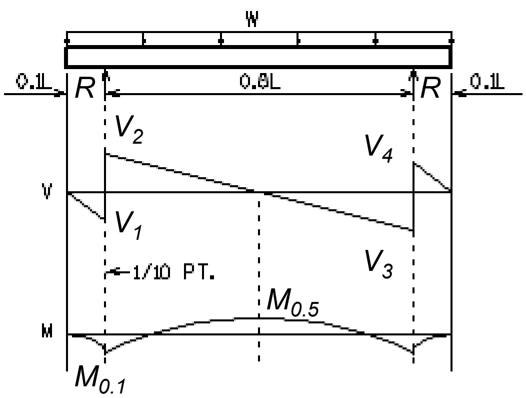
$$f_{top\_prestr} := f_{i\_top\_0.1} - \frac{M_{gird0.1} \cdot 12}{S_{top}}$$ $f_{top\_prestr} = -0.345$ ksi

$$f_{bot\_prestr} := f_{i\_bot\_0.1} - \frac{M_{gird0.1} \cdot 12}{S_{bot}}$$ $f_{bot\_prestr} = 4.07$ ksi

The girder dead load moment and resulting stresses are calculated based on the girder being supported at the lift points. The resulting stresses are added to the stresses due to the prestress forces to give the total stresses during girder picks.

Moments and Shears at the Lift Points, 1/10 point, due to the girder self weight.

$R = 70.05$ kips



$V'_1 := -w_{girder} \cdot 0.1 \cdot L_g$ $\quad V'_1 = -14.01$ kips

$V'_2 := V'_1 + R$ $\quad V'_2 = 56.04$ kips

$V'_3 := V'_2 - (w_{girder} \cdot 0.8 \cdot L_g)$ $\quad V'_3 = -56.04$ kips

$V'_4 := V'_3 + R$ $\quad V'_4 = 14.01$ kips

$$M_{gird0.1\_Lift} := \frac{1}{2} \cdot V'_1 \cdot (L_g \cdot 0.1)$$ $M_{gird0.1\_Lift} = -102.97$ kip-ft

Top of girder stresses due to lifting forces (positive stress values indicate compression.):

$$f_{top\_Lift} := f_{top\_prestr} + \frac{M_{gird0.1\_Lift} \cdot 12}{S_{top}}$$ $f_{top\_Lift} = -0.415$ ksi

$$f_{bot\_Lift} := f_{bot\_prestr} + \frac{M_{gird0.1\_Lift} \cdot 12}{S_{bot}}$$ $f_{bot\_Lift} = 4.135$ ksi

E19-4.3 Check Compression Stresses due to Lifting

Check temporary allowable stress (compression) **LRFD [5.9.4.1.1]**:

$f_{ciall} := 0.65 \cdot f'_{ci}$ where f'_{ci} = 6.8 ksi $f_{ciall} = 4.42$ ksi

Is the stress at the bottom of the girder less than the allowable? $check_{f\_bot} =$ "OK"

If stress at the bottom of girder is greater than allowable, calculate f'_{ci_reqd} :

$$f'_{ci\_reqd} := \frac{f_{bot\_Lift}}{0.65}$$ (not calculated since check is "OK")

E19-4.4 Check Tension Stresses due to Lifting

The temporary allowable tension, from **LRFD [Table 5.9.4.1.2-1]**, is:

$f_{tall} := -0.24 \cdot \lambda \cdot \sqrt{f'_{ci}}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $f_{tall} = -0.626$ ksi

$f_{top\_Lift} = -0.415$ ksi

Is the stress at the top of the girder less than the allowable? $check_{f\_top} =$ "OK"

Therefore, proportion the reinforcement in the top flange using an allowable stress of:

$f_s := min\left(0.5 \cdot f_y, 30\right)$ $f_s = 30$ ksi

E19-4.5 Design Top Flange Reinforcement

Calculate the location of the neutral axis:

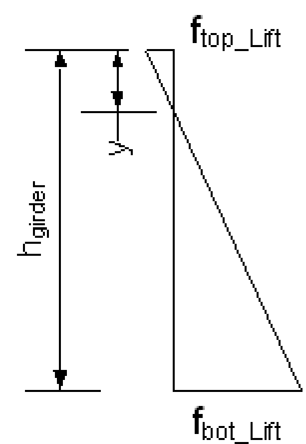
$h_{girder} = 72$ in

$f_{top\_Lift} = -0.415$ ksi

$f_{bot\_Lift} = 4.135$ ksi

$$y := h_{girder} \cdot \frac{f_{top\_Lift}}{f_{top\_Lift} - f_{bot\_Lift}} = 6.56 \text{ in}$$

$y_{Location} =$ "Y is located in the girder web."



Calculate the average flange thickness:

$A_1 := \frac{1}{2} \cdot (t_{top\_flg\_min} + t_{top\_flg\_max}) \cdot (w_{top\_flg} - t_w)$ $A_1 = 176.38$ in²

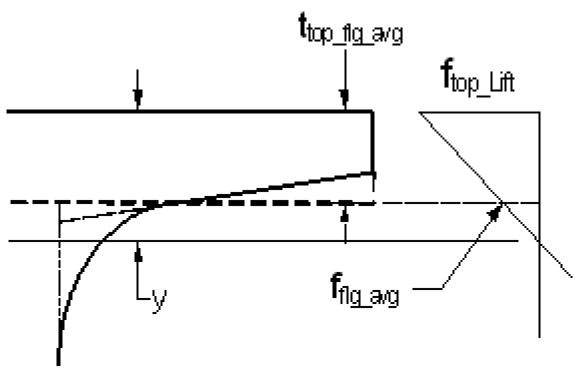
$t_1 := \frac{1}{2} \cdot (t_{top\_flg\_min} + t_{top\_flg\_max})$ $t_1 = 4.25$ in

$A_2 := t_{top\_flg\_max} \cdot t_w$ $A_2 = 35.75$ in²

$t_2 := t_{top\_flg\_max}$ $t_2 = 5.5$ in

$t_{top\_flg\_avg} := \frac{A_1 \cdot t_1 + A_2 \cdot t_2}{A_1 + A_2}$ $t_{top\_flg\_avg} = 4.46$ in

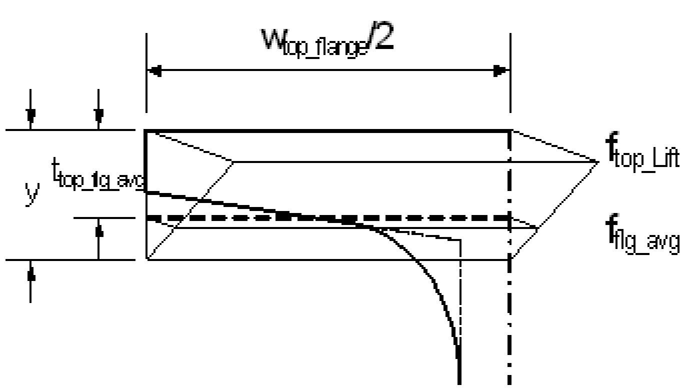
Determine the values of the stress at the average flange thickness.



At $t_{top\_flg\_avg} = 4.461$ inches from the top of the girder:

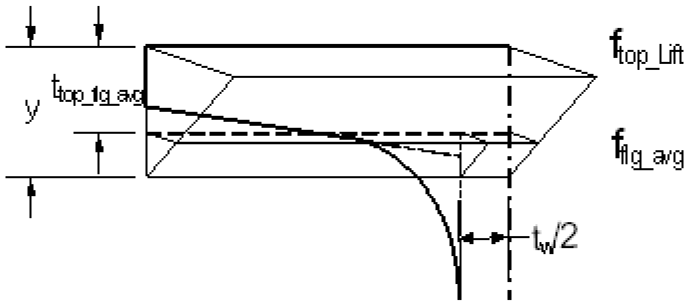
$f_{flg\_avg} := \frac{f_{top\_Lift}}{y} \cdot (y - t_{top\_flg\_avg})$ $f_{flg\_avg} = -0.133$ ksi

Calculate the tension force in the girder flange.



$T_{flg\_avg} := \frac{1}{2} \cdot (f_{top\_Lift} + f_{flg\_avg}) \cdot t_{top\_flg\_avg} \cdot w_{top\_flg}$ $T_{flg\_avg} = -58.65$ kips

Calculate the tension force in the girder web (this minor force can be ignored for simplification).



$T_{web} := \frac{1}{2} \cdot f_{flg\_avg} \cdot (y - t_{top\_flg\_avg}) \cdot t_w$ $T_{web} = -0.91$ kips

$T_{total} := T_{flg\_avg} + T_{web}$ $T_{total} = -59.56$ kips

$T = 59.56$ kips

$As_{Reqd} := \frac{T}{f_s}$ $As_{Reqd} = 1.99$ in²

Use 6 bars in the Top Flange: $Number\_Bars := 6$

Try #6 Bars: $Bar_{No} \equiv 6$

$A_s := \frac{As_{Reqd}}{Number\_Bars}$ $A_s = 0.33$ in² per bar

Area of a #6 Bar: $Bar_A(Bar_{No}) = 0.44$ in² per bar

Is the area of steel per bar greater than required? $check_{As} =$ "OK"

Therefore, use 6 - #6 Bars in Top Flange of Girder for 0.1 point lifting locations.

Note that these bars should be terminated where no longer required by design and lapped with 6 #4 bars as shown on the Standard Details.

Table of Contents

39.1 General

39.1.1 Introduction

Signing is an integral part of the highway plan and as such is developed with the roadway and bridge design. Sign support structures are divided into two categories: Roadside signs, and Overhead Sign Structures (OSS). Roadside signs are designed and specified by the roadway engineer. OSS are designed by a Department (in-house or consultant) structural engineer or by a contractor, depending on the type of OSS.

Generally, an OSS is comprised of three components: the sign(s), the structure, and the foundation. Signage details are provided in the WisDOT Sign Plate Manual referenced below. This chapter of the WisDOT Bridge Manual (BM) governs the design of the structure and the foundation for OSS.

Regional traffic engineers determine the type of overhead sign structure that meets the signage needs for a particular project. Selection guidance and information is provided in the Facilities Development Manual (FDM) 11-55-20. That selection is communicated to the Bureau of Structures through the SSR submittal process.

The responsibility for developing contract plans depends on the type of sign structure selected and may be the role of Bureau of Structure staff, Regional staff, or engineering consultants.

39.1.2 Sign Structure Types and Definitions

Roadside Sign: Refers to roadside signs supported on ground mounted posts adjacent to roadways. Ground mounted sign support posts are not considered “structures” and as such, are not assigned a structure number. See WisDOT Sign Plate Manual for details.

https://wisconsindot.gov/Pages/doing-bus/local-gov/traffic-ops/manuals-and-standards/signplate/signplate.aspx

Overhead Sign Structure (OSS): Refers to structural supports for mounting signs over a roadway. OSS are assigned a structure number and inventoried in WisDOT’s Highway Structures Information (HSI) system. These structures are included on the section 8 structure sheets of a contract plan set.

In prior editions of the Bridge Manual there were two categories of overhead sign structures - “Sign Bridges” and “Overhead Sign Supports (OHSS)”. Sign bridges were Department designed, and OHSS were contractor designed. While the roles of design remain the same, this edition shifts away from that terminology, instead focusing on terminology that best describes the geometric characteristics of the sign structure.

Table 39.1-1 summarize OSS types used by WisDOT:

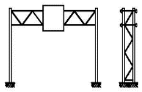
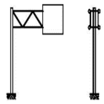
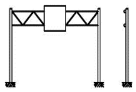
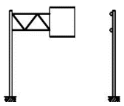
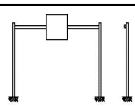
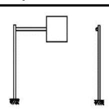
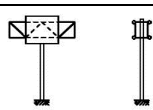
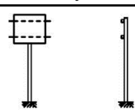
Overhead Sign Structure Type	Description	Standard Structure Design	Standard Foundation Design
 Full Span 4-Chord Truss	A 4-chord space truss with dual, trussed vertical support posts at each end. Used to support large Type I static highway sign panels and Dynamic Message Signs (DMS). Typically used over multi-lane state highways and interstate routes.	Yes	Yes
 Cantilever 4-Chord Truss	A 4-Chord space truss with a single vertical support post. Used to support large Type I static highway sign panels and DMS. Commonly used to span over the outside lanes of multi-lane state highways and interstate routes to delineate exit lanes and ramps.	Yes	Yes
 Full Span 2-Chord Truss	A 2-chord planar truss with single vertical support posts at each end. Used to support Type II and smaller Type I static signs and DMS over roadways and state highways.	No	Yes
 Cantilever 2-Chord Truss	A 2-chord planar truss with a single vertical support post. Used to support Type II and smaller Type I static signs and DMS over roadways and state highways.	No	Yes
 Full Span Monotube	Similar to a Full Span 2-Chord Truss but with only a single horizontal sign support member. Used to support small Type II static signs.	No	Yes
 Cantilever Monotube	Similar to a Cantilever 2 -Chord Truss but with only a single horizontal support member. Used to support small Type II static signs.	No	Yes
 Butterfly Truss	A 4-Chord space truss with a centrally located single vertical support post used to support DMS. Typically used in the medians of multi-lane interstate routes.	No	No
 Butterfly	Similar to a Butterfly Truss but with multiple monotube horizontal sign support members. Structures may include a light pole attached to the top of the column.	Yes	Yes
 Bridge Mounted Sign Support	Sign support brackets to mount signs to the sides of grade separation highway bridges over the underpass roadway. These are typically used in special circumstances where other OSS types cannot be used.	No	NA

Table 39.1-1

WisDOT Overhead Sign Structure Types

39.1.3 Additional Terms

Type I Sign: Larger signs on an extruded aluminum base material, typically mounted on steel I-beams. Large guide and message signs with green backgrounds on interstate routes are Type I signs.

Type II Sign: Signs consisting of direct applied message on either plywood or sheet aluminum base material, typically mounted on wood or steel posts.

Dynamic Message Sign (DMS): An electronic traffic sign, often used in urban settings to inform drivers of unique and variable information. These signs are generally smaller in wind loaded area than Type I signs, but are heavier and load the truss eccentrically.

OSS Standard Designs: A group of pre-designed sign structures. The standard design includes both the structure and its foundation. The limitations for use is provided in section 39.1.5 and 39.1.6. See for further information on OSS Standard Designs.

OSS Non-Standard Design: Refers to sign structures that fall outside the OSS Standard Design parameters. It also applies to sign structure types not covered by standard design. These sign structures require a structural engineer provide a unique individual design of the structure and/or its foundation. See 39.4.5 for further information on OSS Non-standard Designs.

OSS Contractor Designed: Refer to sign structures (including anchor rods) that are designed and detailed by the contractor as part of the construction contract. The limitations for use is provided in sections 39.1.5 and 39.1.6. The contractor does not design the foundation. For this, pre-designed foundations are available for use with these types of sign structures. See 39.4.6 for further information on OSS Contractor Designed.

OSS Standard Design Drawings: Refers to a library of WisDOT developed detail drawings for the *OSS Standard Designs* and the foundations for *OSS Contractor Designed*, otherwise indicated by a “yes” in Table 39.1-1. These standard design drawings are inserted into the contract plans with no additional design or detailing effort required.

39.1.4 OSS Selection Criteria

Chapter 11-55-20 of the Facilities Development Manual (FDM) provides selection guidance for determining sign structure type. The selection guidance was developed based on the design limitations of Table 39.1-1 and Table 39.1-2, and the information provided in the OSS Standard Design Drawings.

39.1.5 Cantilever OSS Selection Criteria

Cantilever OSS Type	Design	Cantilever Length [1]	Vertical Support Height [1]	Static Sign Total Area & Max. Dimensions [2]		DMS Total Area & Weight [1]
Monotube	Contractor Designed	40'-0" Max.	25'-0" Max. Column Base Plate to CL of Monotube Arm	Sign Area ≤ 75 SF Max. Sign Height ≤ 5'-0"	OR	Not Used
2-Chord Truss	Contractor Designed	40'-0" Max. (static) / 20'-0" Max. (DMS)	27'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 150 SF Max. Sign Height ≤ 10'-0"		13'-9"W x 8'-0"H Max. 750 Lbs. Max
4-Chord Truss	Standard Design	20'-0" Min. 30'- 0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 264 SF Max. Sign Height ≤ 15'-0"		19'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Standard Design	>30'-0" 38'-0" Max.[2]	30'-0" Max. Column Base Plate to CL of Top Chord	Sign Area ≤ 240 SF Max. Sign Height ≤ 15'-0"		19'-0"W x 6'-0"H 2,500 Lbs. Max.
4-Chord Truss	Non-Standard Design	>38'-0"	Column Height Exceeds Limit for Standard Design	Sign Area or Max. Sign Height Exceeds Limits For Standard Design		DMS Dimensions or Weight Exceeds Limits For Standard Design

Table 39.1-2
Cantilever OSS Selection Criteria

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

Note 2: Static Type I sign panels may extend 1'-0" beyond end of Cantilever 4-Chord Truss.

39.1.6 Full Span OSS Selection Criteria

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

Table 39.1-3
Full Span OSS Selection Criteria

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

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

39.1.7 Butterfly and Butterfly Truss OSS

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

Table 39.1-4
Butterfly and Butterfly Truss OSS Selection Criteria

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

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

39.1.8 Design Process

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

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

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

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

39.4 Design Considerations

39.4.1 Roadside Signs

Supports for roadside signs are of two types, depending upon the size and type of the sign to be supported. For small signs, the column supports are treated timber embedded in the ground. For larger Type 1 signs and DMS, the columns are galvanized steel supported on drilled shafts. Standard design and support estimates are given in the A3 Series of the "Sign Plate Manual."

Roadside sign supports are located behind existing or planned guardrail as far as practical from the roadway and out of the likely path of an errant vehicle. If roadside signs are located within the 30-foot corridor and not protected, break-away sign supports are detailed. Roadside sign supports for DMS, which includes dynamic message signs and variable message signs, are to be protected by concrete barrier or guardrail.

Currently, all steel column supports for roadside Type 1 signs, and DMS are designed to break-away upon impact.

The Wisconsin DOT Bureau of Traffic Operations has standard designs and details available for Type 1 Roadside Sign supports and foundations. The standard steel post design tables provide maximum sign mounting heights. If a sign configuration is required that does not fall within the limits of the standard designs, the sign support must be designed by a structural engineer. The design must be in compliance with the applicable specifications listed in 39.3. The Type 1 roadside sign standard foundation designs are based on the assumptions of cohesionless soils with the following properties:

- Soil Unit Weight = 115 pounds per cubic foot
- Angle of Internal Friction = 24 degrees
- Soil Modulus Parameter = 25 pounds per cubic inch

Wisconsin has standard design and details available for DMS roadside sign supports. If weaker subsurface conditions are known or suspected, a subsurface soil investigation per 39.5 would be implemented to gather necessary design information.

39.4.2 Overhead Sign Structures (OSS)

39.4.2.1 General

OSS types and names used by WisDOT are summarized in Table 39.1-1.

The connections of web members to chords are designed for bolted or shop welded connections to allow the contractor the option to either galvanize individual members or complete truss sections after fabrication. Steel base plates are used for anchor rod support attachment.

Aluminum sign structures are currently not being designed for new structures. Rehabilitation and repair type work may require use of aluminum members and shall be allowed in these limited instances. The following guidelines apply to aluminum structures in the event of repair and rehabilitation type work.

Aluminum sign structure trusses are designed and fabricated from tubular shapes shop welded together in sections. The minimum thickness of truss chords is 1/4-inch and the minimum outside diameter is 4 inches. The recommended minimum ratios of “d/D” between the outside diameter “d” of the web members and “D” of chord members is 0.4. A cast aluminum base plate is required to connect the aluminum columns to the anchor rods. AASHTO Specifications require damping or energy absorbing devices on aluminum overhead sign support structures to prevent vibrations from causing fatigue failures. Damping devices are required before and after the sign panels are erected on all aluminum sign bridges. Stock-bridge type dampers are recommended.

39.4.2.2 Vehicular Protection

Vertical supports for OSS Standard Designs are not designed for vehicular impact loads and must meet clear zone or barrier protection requirements in the FDM. Generally, all overhead sign structure vertical supports are located at the edge of shoulder adjacent to the traveled roadway and placed behind roadside concrete barriers or barrier type guardrail. See the FDM 11-55-20.6 for information on shielding requirements. Sign supports protected by roadside barriers or guardrail with adequate barrier deflection clearance between the backside of the barrier and the sign support are not required to be designed for Extreme Limit State vehicular collision loads.

When protection is not feasible, the vertical supports shall be designed with applicable Extreme Event collision loads in accordance to 13.4.10. This typically requires the use of a special, individually designed reinforced column and foundation to resist the large vehicular impact loads. In this situation the sign structure would be a non-standard design and BOS or an engineering consultant would need to provide the design.

39.4.2.3 Vertical Clearance

As provided in the FDM 11-35-1 Attachment 1.8, a minimum vertical clearance of 18’-3” is required for most routes. For sign structures over a designated High Clearance Route, 20’-3” above the roadway is required. See FDM 11-35-1 Attachment 1.9 for clearances relating to existing sign structures.

39.4.2.4 Lighting and DMS Inspection Catwalks

Lighting is no longer required on sign structures. Catwalks are only on 4-chord cantilever and full span OSS with DMS. When catwalks are provided for OSS supporting a DMS, additional vertical height must be provided to meet the vertical clearance requirements in 39.4.2.3 to the bottom of the catwalk brackets. Catwalk grating and toe plates shall be galvanized steel.

Along with inspection catwalks, all DMS OSS require hand holes, rodent screens and electrical conduits through the foundation to one of the vertical support posts to route electric power to the DMS. Standard Details are provided on the BOS website.

39.4.2.5 Signs Mounted on the Side of Grade Separation Bridges

When no practical alternatives exist, signs may be mounted on the side of grade separation bridges. This application requires individually designed structural mounting brackets to attach the sign to the side of the grade separation bridge. Wisconsin allows sign attachments orientated up to a maximum of a 20-degree skew to the roadway below. Any grade separation bridge structure with greater skew requires the mounting brackets to attach signs so they are orientated perpendicular to the roadway below.

Where possible, the depth of bridge mounted signs should be limited so the top of the sign does not extend above the top of the bridge parapets or railing. Signs are not permitted to extend below the bottom of the bridge girders. The minimum vertical clearance is set slightly above the clearance line of the overpass structure for signs attached directly to the side of a bridge.

Signs attached to the side of bridge superstructures and retaining walls are difficult to inspect and maintain due to lack of access. Attachment accessories are susceptible to deterioration from de-icing chemicals, debris collection and moisture. Therefore, the following guidance should be considered when detailing structure mounted signs and related connections:

1. Limit the sign depth to a dimension equal to the bridge superstructure depth (including parapet) minus 3 inches.

2. Provide at least two support connections per bracket.

3. Utilize cast-in-place anchor assemblies to attach sign supports onto new bridges and retaining walls.

4. Galvanized or stainless-steel adhesive concrete masonry anchor may be used to attach new signs to the side of an existing grade separation bridge or retaining wall orientated for shear load application only. Overhead anchor installation (direct pullout loading on anchor) is not allowed. Reference 40.16 for applicable concrete masonry anchor requirements.

39.4.2.6 Sign Structures Mounted on Bridge Pedestals

This refers to sign structures mounted across the top of roadways carried by a bridge structure. Sign structures can be mounted directly to the top of pier caps. This requires the pier cap to be extended beyond the limits of the superstructure width. Sign structures mounted to pier caps are not affected by superstructure deflections. Wisconsin allows sign attachments oriented up to a maximum of a 20-degree skew to the roadway below. Any grade separation bridge structure with a greater skew requires the mounting brackets to attach signs so they are oriented perpendicular to the roadway below.

Span live load deflections of the vehicular bridge superstructure affect sign structures mounted on to bridge superstructure concrete barrier pedestals. The magnitude of sign structure deflections and duration of sign structure vibrations is dependent on the stiffness of the girder and deck superstructure, the location of the sign structure on the bridge, and the ability of the sign structure to dampen those vibrations out; among others. These vibrations are not easily accounted for in design and are quite variable in nature. For these reasons, the practice of locating sign structures on highway bridges should be avoided whenever possible.

The following general guidance is given for those instances where locating a sign structure on a bridge structure is unavoidable. This may occur due to the length of the bridge, or a safety need to guide the traveling public to upcoming ramp exits or into specific lanes on the bridge.

1. Locate the sign structure pedestals at pier locations.

2. Build the sign structure base off the top of the pier cap.

3. Provide adequate set back of the tower support of the sign structure behind the face of the parapet to avoid snagging of vehicles making contact with the parapet. See FDM 11-45-2.3.6.2.3 for information on required set back distances.

4. Use single pole sign supports (equal balanced butterfly's) in lieu of cantilevered (with an arm on only one side of the vertical support) sign supports.

5. Consider the use of a Stockbridge type damper in the horizontal truss of these structures.

6. Do not straddle the pier leaving one support on the pier and one support off the pier in the case of skewed substructure units for full span sign bridges

39.4.3 LRFD Requirements and WisDOT Guidance for OSS Design

39.4.3.1 Loads, Load Combinations, and Limit States

All OSS are to be designed per the AASHTO LRFDLTS-1. The following LRFD specification requirements are highlighted:

Design Wind Speed Recurrence Interval:

- Full Span 4-Chord Truss and median Butterfly Sign Structures are designed for a basic wind speed recurrence interval of 1,700 years as defined in the AASHTO LRFDLTS-1 Specifications.

- All other OSS shall be designed for a basic wind speed recurrence interval of 700 years as defined in the AASHTO LRFDLTS-1 Specifications.

Wind load and wind load combinations shall be applied and investigated per AASHTO LRFDLTS-1. In general, horizontal wind pressure is applied normal to the center of gravity of exposed horizontal members and sign panels. For the design of vertical supports, three wind load cases are investigated and applied to the entire structure to determine the controlling wind load effect on the vertical supports:

Wind Load Case	Description	Normal Wind Component	Transverse Wind Component
1	Full Wind Normal to the Plane of the Structure	100%	0%
2	Full Wind Transverse to the Plane of the Structure	0%	100%
3	75% Full Wind in Both Directions Simultaneously	75%	75%

Figure 39.4-1
AASHTO LRFDLTS-1 Vertical Support Load Cases

Design sign area assumed for standard designs accommodates 12-foot high sign panels over 90% of the span length for full span 4-chord truss OSS. In the case of a proposed non-standard OSS with a required span length of 130 feet or less, the non-standard OSS should be individually designed for the actual anticipated sign panel area or DMS and mounting locations. In the case of a proposed non-standard OSS with required span length greater than 130 feet, the Bureau of Structures should be consulted to confirm the design sign area to be used for the design of the non-standard OSS.

Applied design wind pressure is determined for individual truss members and sign panels per the AASHTO LRFDLTS-1 specification Section 3.8. WisDOT design practice is to assume members located directly behind sign or DMS panels are shielded from wind exposure and are not loaded with wind pressure. No wind shielding is assumed provided to members that directly align with each other in plan or elevation views, but are several feet apart. This means no shielding effect is assumed for members in the front and back truss planes of a 4-chord truss even if the members are perfectly aligned. For example, viewing a 4-chord truss in elevation view, members in the front truss plane, located directly behind a sign panel would be assumed to be shielded from wind pressure by the sign panel, but members in the back-truss plane would be assumed to be loaded with wind pressure, despite also being behind the sign panel or aligned with other members in the front truss plane.

Strength 1 load combinations in AASHTO LRFDLTS-1 include only dead load and live load. A 500-pound live load distributed over 2’-0” transversely to the member, only applies to catwalks and catwalk support brackets when catwalks are included for OSS with DMS. The Strength 1 load combination may control the design of the catwalk and catwalk support brackets, but does not control the design of the truss superstructure the catwalk brackets are attached to. For OSS carrying static Type 1 signs, the Strength I load combination includes only dead load and does not control.

Load combinations that include wind generally control the design of sign structures. A change in the AASHTO LRFDLTS-1 specification is that load combinations that include wind are considered Extreme Event load cases.

AASHTO LRFDLTS-1 specifications do not define an ice loading and leave it to the discretion of individual owners to consider and specify an ice loading if warranted in their climate. WisDOT policy is to maintain consideration of an ice load and include in the Extreme Event I load combination.

Load Combinations are as follows:

Strength I:	1.25 DL + 1.6 LL
Extreme Event I (Load Case 1):	1.1 DL + 1.0 ICE + W (Max. DL and ICE effects)
Extreme Event I (Load Case 2):	0.9 DL + W (Min. DL and no ICE effects)

Ice build-up is modeled as a 3 psf load applied to the exposed surface area (circumference) of truss members. It is not necessary to increase the wind pressure load on truss members due to increased member exposure area caused by ice build-up. Ice load is applied to only the front face of sign or DMS panels.

For vertical column support members, W in the above Extreme Event load cases is the controlling wind load case as specified in Figure 39.4-1.

39.4.3.2 Serviceability

Serviceability checks should conform to 10.4 and 10.5 of AASHTO LRFDLTS-1. However, the vertical deflection limit L/150 shall include ICE load, applied per 39.4.3.1.

39.4.3.3 Fatigue

AASHTO LRFDLTS-1 specifies three fatigue loads to check against member and connection fatigue stress range limits as follows:

Galloping – AASHTO LRFDLTS 11.7.1.1:	Applies to all cantilever OSS, except cantilever 4-chord truss
Natural Wind Gust – AASHTO LRFDLTS 11.7.1.2:	Applies to all OSS.
Truck Induced Gust – AASHTO LRFDLTS 11.7.1.3:	Applies to all OSS.

Truck induced gust pressure is applied in the upward direction and reduces with increasing height. Truck induced gust pressure applied to truss members in the top horizontal truss plane, will be less than truck induced gust pressures applied to truss members in the bottom horizontal truss plane. Since truck induced gust pressure is acting upward, Type 1 static signs receive and transmit only minimal gust pressure due to their narrow profile when viewed in plan. DMS however, have considerable width and “wind exposure area” when viewed in plan. Truck induced gust pressure can impart a significant upward pressure on DMS that also creates a torque on the truss superstructure due to the offset between the center of gravity of the DMS and the truss superstructure.

39.4.3.4 Connection Design

WisDOT policy item:

Bureau of Structures policy is to design welded and bolted connections per the applicable provisions of the current AASHTO LRFD Bridge Design Specifications. This is a deviation from the AASHTO LRFDLTS-1, which refer the design of welded connections to the AWS D1.1 Structural Welding Code.

For truss superstructures, current practice is to design and provide alternate details of the connection of web members (angles) to main chord members (HSS tubular round sections) for both welded and bolted connections, except the chord to column connection and first panel of cantilever trusses which must be bolted. This affords the fabricator the option of galvanizing individual members prior to truss fabrication (using bolted connections) or galvanizing entire truss segments after assembly (using bolted or welded connections).

39.4.4 OSS Standard Designs

Standard Design OSS types and limitations are listed in Table 39.1-2 and Table 39.1-3. Because these structures are pre-designed and pre-detailed the involvement of a Department structural engineer is usually not required. Bureau of Structures is responsible for maintaining and updating the Standard Designs as needed.

The Standard Design OSS types were developed to cover a wide range of signage requirements while placed over typical roadway and roadside configurations. Standard Designs are not intended to cover unique situations or unusual geometry, or for reasons described in 39.4.5. Contact the Bureau of Structures Design Section with questions regarding applicability of standard designs.

Standard Foundation Designs are included in the OSS Standard Detail Drawings. See 39.5 for further guidance on subsurface investigation, assumed soil parameters, and foundation design.

When Standard Design OSS are used in a project, the Region or their consultant prepare final contract plans as described in 39.1.9. This includes filling out the appropriate sign structure specific information on both the General Notes and General Layout sheets and combining those with the appropriate OSS Standard Design Drawings to make up the final plan set.

39.4.5 OSS Non-Standard Designs

Design and plan detailing must be provided by Bureau of Structures or by a structural design consultant for all non-standard designs. The following circumstances warrant a non-standard design:

1. The OSS type is Butterfly Truss or Bridge Mounted

2. The OSS type falls outside the limits of span length, sign area, DMS weight, or sign height in FDM 11-55-20 Figure 20.2.3 and Figure 20.2.4.

3. Region soil engineer advises that subsurface conditions at the site are expected to negatively differ from assumed soil profile and design parameters of standard foundations (e.g. soft soil or shallow bedrock – see 39.5.2.2).

4. Excessive sign structure height (e.g. sign structure behind MSE wall) or requires the use of concrete column (designed for impact load – see 39.4.2.2)

BOS must be consulted to verify and confirm the need for individual designs before undertaking this effort.

The design detailing shall generally follow the guidance provided by the OSS Standard Design Drawings but should clearly delineate any required changes to individual member sizes, connections and foundation details necessary to satisfy the AASHTO LRFDLTS-1 Design Specifications.

In some instances, it may still be appropriate to use part or all of the Standard Designs even though the sign structure is considered a Non-standard Design. A couple of examples include:

1. A sign structure has both static and DMS sign types specified for mounting (consult with BOS before using a standard design in this situation).

2. A Standard Design structure is used in conjunction with a Non-standard foundation. See section 39.5.3.

In any case, the sign structure is still considered a Non-standard design in terms of the design process and should proceed as detailed in 39.1.9.

39.4.6 OSS Contractor Designed

Contractor Designed OSS types and limitations are listed in Table 39.1-2 and Table 39.1-3. Because these structures are designed by the contractor, involvement of a Department structural engineer is usually not required.

Standard Foundation Designs are included in the OSS Standard Detail Drawings. The standard foundations only include the design of the drilled shaft, the contractor is responsible for designing the anchor rods and superstructure. See 39.5 for further guidance on subsurface investigation, assumed soil parameters, and foundation design. Bureau of Structures is responsible for maintaining and updating the standard foundation designs that go along with the Contractor Designed OSS types.

These structures are designed for the required actual sign area and configuration, unless future expansion is anticipated, which should be noted and shown on the plans. The required actual sign area, span length, etc. is used to select the appropriate standard foundation from the figure provided in chapter 11-55-20 of the FDM.

When Contractor Designed OSS are used in a project, the Region or their consultant prepare final contract plans as described in 39.1.9. This includes filling out the appropriate sign structure specific information on both the General Notes and General Layout sheets and combining those with the appropriate OSS Standard Design Drawings to make up the final plan set.

39.5 Geotechnical Guidelines

39.5.1 General

For butterfly structures and 4-chord trusses, the typical preferred foundation is comprised of two cylindrical drilled shafts connected by a concrete cross-girder, as detailed in the OSS Standard Design Drawings. The top of the cross-girder is set 3 feet above the highest ground elevation at the foundation. For all other types, the typical preferred foundation is comprised of a single cylindrical drilled shaft directly supporting the column vertical support. Occasionally, some columns are mounted directly on top of modified bridge parapets, pier caps and concrete towers instead of footings.

There are several potential challenges regarding subsurface exploration for OSS foundations:

- The development and location of these structures are typically not known at the onset of the preliminary design stage, when the most subsurface exploration typically occurs. This creates the potential need for multiple drilling mobilizations for the project.

- OSS are often located in areas of proposed fill soils. The source and characteristics of fill soil is unknown at the time of design.

- OSS foundations are often located on the shoulder or median directly adjacent to high-volume roadways. Obtaining boings in these locations typically requires significant traffic control, night work, and working in a potentially hazardous work zone.

- If a consultant is involved in the project, the unknowns associated with these structures in the project scoping stage complicate the consultant contracting process. It is often difficult to determine the need for OSS specific subsurface investigation at the time the consultant contract is normally being scoped. In cases where the need for a specific subsurface investigation is known or anticipated, an assumption must be made regarding the level of subsurface investigation to include in the consultant design contract. Alternatively, a decision can be made to assume use of standard OSS and foundation designs. If the need for specific subsurface investigation is later determined to be necessary, this may require a change to add it to the consultant contract.

39.5.2 Standard Foundations for OSS

39.5.2.1 General

WisDOT has created standard full span and cantilever 4-chord truss designs that include fully designed and detailed drilled shaft foundations as part of the overall standard design. The standard foundation details are incorporated with the OSS Standard Design Drawings for these structures and are available on the BOS website.

Single drilled shaft OSS Standard Design Drawings for use with contractor designed full span and cantilever 2-chord truss and monotube OSS are also available on the BOS website.

WisDOT has no standard foundation design details for alternate foundation types and the selected alternative foundations would be required to be individually designed and reviewed by BOS.

39.5.2.2 Design Parameters Used for Standard Foundation Design

Standard dual and single drilled shaft foundation designs were developed in accordance with applicable requirements of Section 10 of the *AASHTO LRFD Bridge Design Specifications.*

The standard foundation designs are based on the following design parameters:

- Total Unit Weight = 125 pcf
- Granular Soil Profile: Internal Angle of Friction = 24 degrees, or
- Cohesive Soil Profile: Undrained Shear Strength = 750 psf
- Soil and drilled shaft downward resistance factor ϕ =1.0 [1]
- Drilled shaft uplift resistance factor ϕ =0.8 [1]
- Depth of water table assumed 10 feet below the ground surface
- Soil side resistance is considered fully effective to the top of the drilled shaft or top of ground surface, whichever is the lower elevation.
- Lateral deflection at the top of the foundation limited to 1-inch at the Service I Limit State

Note 1: Resistance factors per AASHTO 10.5.3.3 assuming the drilled shaft design is governed by the wind load combination which is an Extreme Event load combination.

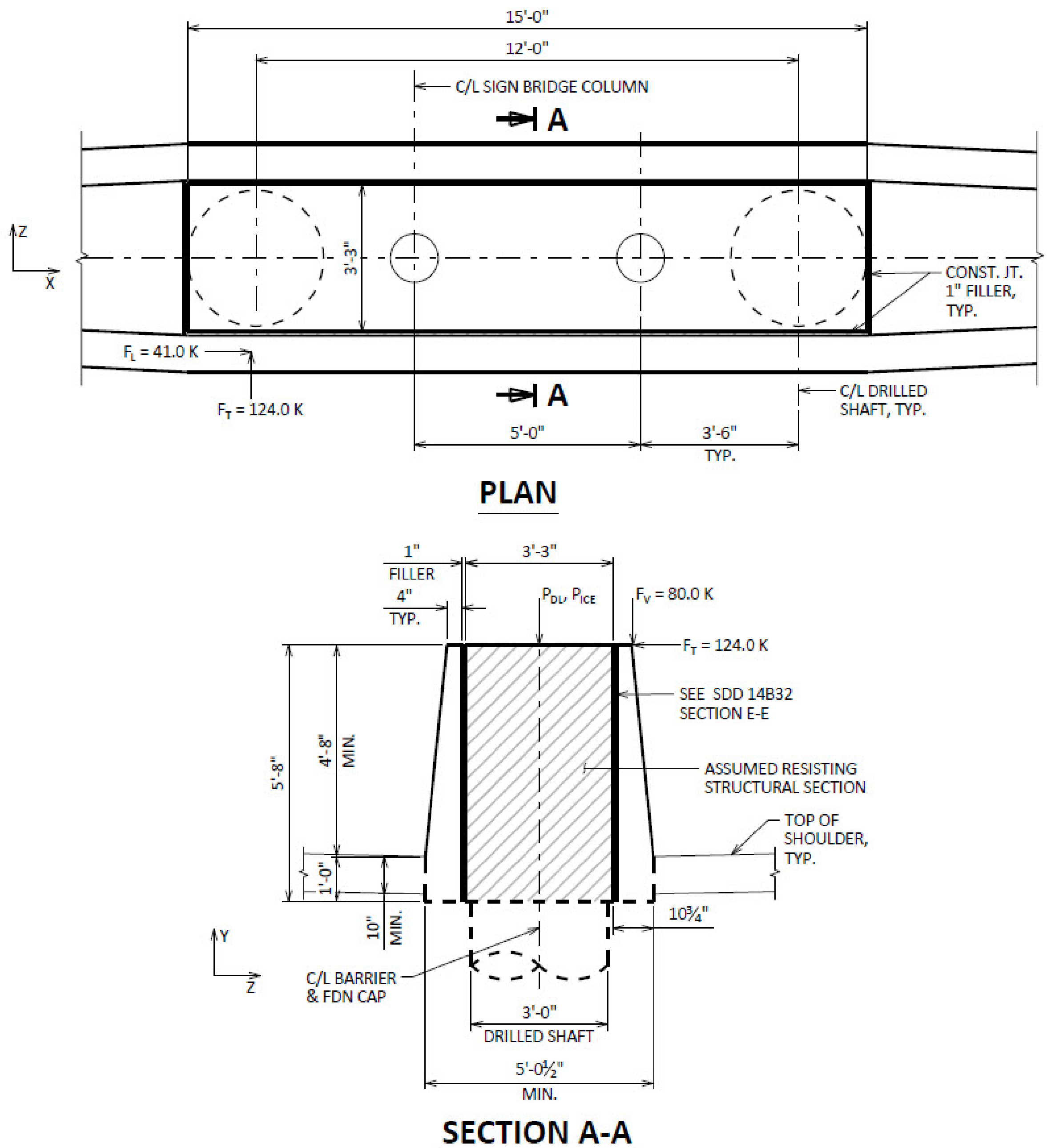
WisDOT policy item:
Design of standard sign structure foundations assumes soil side resistance is fully effective to the top of the drilled shafts for full span 4-chord OSS foundations and to within 3 feet below the lowest ground surface for all other OSS foundations. This is a deviation from AASHTO 10.8.3.5 1b.

Use of the standard foundations requires that the in-situ soils parameters at the site meet or exceed the assumed soil design parameters noted above. Soil parameters were selected to be sufficiently conservative to cover most sites across the state. Designers should contact the Region Soils Engineer or the Geotechnical Consultant to assist in the evaluation of the subsurface conditions compared to the assumed soil parameters. An assessment can also be made by checking nearby borings and as-built drawings of nearby existing structures, and similar sources. If there is reason to suspect weaker soils or that shallow bedrock is present, OSS specific soil borings should be obtained to confirm in-situ soil properties meet or exceed the assumed parameters used for the standard designs. If these site-specific soil properties

Table of Contents

E39-1 Design of Foundation Cap Beam / Integral Barrier - TL-5 Loading

This example shows design calculations for a four chord sign bridge foundation cap beam supported on two drilled shafts that is integral with a roadway barrier. The *AASHTO LRFD Bridge Design Specifications 8th Edition - 2017* are followed for the cap beam design using a TL-5 design force for traffic railings.



E39-1.1 Design Criteria

Cap/Integral Barrier Material Properties

$f'_c := 3.5$ ksi	Concrete Strength
$f_y := 60$ ksi	Yield Strength of Reinforcement
$E_s := 29000$ ksi	Modulus of elasticity of steel
$w_c := 0.150$ kcf	Unit Weight of concrete

Barrier and Foundation Geometry

$H_{barrier} := 66.00$ in	Height of Barrier
$H_{barrier\_vert} := 10.00$ in	Height of Barrier Vertical Section
$W_{barrier\_top} := 49.00$ in	Width of Barrier at Top
$W_{barrier\_bott} := 60.5$ in	Width of Barrier at Bottom
$W_{barrier\_str} := 39.00$ in	Width of Barrier Structural Section
$L_{barrier} := 15.00$ ft	Length of Barrier Section
$Diam_{shaft} := 3.00$ ft	Diameter of Drilled Shaft
$Shaft\_Spa := 12.00$ ft	Spacing Between Drilled Shafts

E39-1.2 Design Forces for Traffic Railings

From **LRFD Table A13.2-1**, use Test Level Five (TL-5) design forces for integral barrier/cap check. Forces are conservatively applied as point loads instead of being distributed longitudinally along the integral barrier/cap foundation length.

$F_t := 124.0$ kips	Transverse design load
$F_L := 41.0$ kips	Longitudinal design load
$F_v := 80.0$ kips	Vertical design load (down)
$H_e := 56.0$ in	Minimum height of transverse design load = 42". Apply transverse load at top of barrier.

E39-1.3 Loads

Barrier/Cap Uniform Dead Load

Note - Uniform Dead Load is for the full area of the integral barrier including portions of the barrier outside the structural section.

$H_{barrier\_slope} := H_{barrier} - H_{barrier\_vert}$ $\qquad H_{barrier\_slope} = 56.00\ in$

$$Area_{barrier} := \left(H_{barrier\_slope} \cdot \text{mean}\left(W_{barrier\_top}, W_{barrier\_bott}\right) + H_{barrier\_vert} \cdot W_{barrier\_bott}\right) \cdot \frac{1}{144}$$

$Area_{barrier} = 25.493\ ft^2$

$W_{DC} := Area_{barrier} \cdot w_c$ $\qquad W_{DC} = 3.824$ kips/ft

Sign Structure Dead and Ice Loads - bottom of column reaction taken from SAP2000 analysis for an 82 ft span sign bridge with 30 ft column height.

$P_{dl} := 8.05\ kips$ $\qquad P_{ice} := 3.34\ kips$

Barrier Live Load - There is no live load on the barrier since there is no live load on the sign structure.

E39-1.4 Limit States and Combinations

Limit State Extreme Event II for vehicle collision shall be applied using the following equation and load factors from **LRFD Table 3.4.1-1 & Table 3.4.1-4.**

$M_u := 1.0 \cdot DC + 0.5 \cdot LL + 1.0 \cdot IC + 1.0 \cdot CT$

E39-1.5 Analysis Case I

Maximize moments in integral barrier/foundation cap by placing TL-5 loads at midspan between the drilled shafts. Assume barrier is a simply supported span between the centerlines of the drilled shafts.

Moments due to transverse forces:

$M_{y\_DC} := 0.0$ $\qquad M_{y\_LL} := 0.0$ $\qquad M_{y\_IC} := 0.0$

$M_{y\_CT} := F_t \cdot \dfrac{\left(L_{barrier} - Diam_{shaft}\right)}{4}$ $\qquad M_{y\_CT} = 372.0\ ft \cdot kips$

$M_{uy} := 1.0 \cdot M_{y\_DC} + 0.5 \cdot M_{y\_LL} + 1.0 \cdot M_{y\_IC} + 1.0 \cdot M_{y\_CT}$ $\qquad M_{uy} = 372.0\ ft \cdot kips$

Moments due to vertical forces:

$CL_{col\_shaft} := 4.25 \; ft$ Distance from CL drilled shaft to center of cap.

$$M_{z\_DC} := \frac{W_{DC} \cdot (Shaft\_Spa)^2}{8} + \frac{P_{dl}}{2} \cdot CL_{col\_shaft} \qquad M_{z\_DC} = 85.9 \; kip \cdot ft$$

$$M_{z\_IC} := \frac{P_{ice}}{2} \cdot CL_{col\_shaft} \qquad M_{z\_IC} = 7.1 \; kip \cdot ft$$

$$M_{z\_LL} := 0.0 \qquad M_{z\_LL} = 0 \; kip \cdot ft$$

$$M_{z\_CT} := \frac{F_v \cdot Shaft\_Spa}{4} \qquad M_{z\_CT} = 240.0 \; kip \cdot ft$$

$$M_{uz} := 1.0 \cdot M_{z\_DC} + 0.5 \cdot M_{z\_LL} + 1.0 \cdot M_{z\_IC} + 1.0 \cdot M_{z\_CT} \qquad M_{uz} = 333.0 \; kip \cdot ft$$

E39-1.6 Analysis Case II

Maximize shears in integral barrier/foundation cap by placing TL-5 loads at centerline of drilled shaft. Assume shear is resisted by a single shaft (conservative).

Shears due to transverse forces:

$V_{z\_DC} := 0.0$ $V_{z\_LL} := 0.0$ $V_{z\_IC} := 0$

$$V_{z\_CT} := F_t \qquad V_{z\_CT} = 124.0 \; kips$$

$$V_{uz} := 1.0 \cdot V_{z\_DC} + 0.5 \cdot V_{z\_LL} + 1.0 \cdot V_{z\_IC} + 1.0 \cdot V_{z\_CT} \qquad V_{uz} = 124.0 \; kips$$

Shears due to vertical forces:

$$V_{y\_DC} := P_{dl} + W_{DC} \cdot \frac{L_{barrier}}{2} \qquad V_{y\_DC} = 36.73 \; kips$$

$$V_{y\_IC} := P_{ice} \qquad V_{y\_IC} = 3.34 \; kips$$

$$V_{y\_LL} := 0.0$$

$$V_{y\_CT} := F_v \qquad V_{y\_CT} = 80.00 \; kips$$

$$V_{uy} := 1.0 \cdot V_{y\_DC} + 0.5 \cdot V_{y\_LL} + 1.0 \cdot V_{y\_IC} + 1.0 \cdot V_{y\_CT} \qquad V_{uy} = 120.07 \; kips$$

E39-1.7 Flexural Strength Capacity

For rectangular section behavior (vertical loading):

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$

LRFD [5.6.2.2] $\alpha_1 := 0.85$ $(for \cdot f'_c < 10.0 \cdot ksi)$

$$\beta := \begin{cases} 0.85 & \text{if } f'_c \le 3.5 \\ 0.85 - (f'_c - 4) \cdot 0.05 & \text{else} \end{cases}$$

$\beta_1 := \max(\beta, 0.65)$ $\beta_1 = 0.85$

$b := W_{barrier\_str}$ $b = 39.00\ in$

The 82 ft. span sign bridge with 30 ft. column height standard foundation cap provides #6 bars for bottom reinforcement and #6 bar stirrups. For the vehicular collision force, which is an extreme limit event state not included in the standard foundation cap designs, it is necessary to increase the bottom reinforcement to at least 7 - #7 bars:

$A_{st\_7} := 0.60\ in^2$ $Num\_bars := 7$

$A_s := A_{st\_7} \cdot Num\_bars$ $A_s = 4.20\ in$

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$ $c = 2.56\ in$

$a := \beta_1 \cdot c$ $a = 2.17\ in$

$Clr\_cov := 3.00\ in$ Bottom bar clear cover

$dia_7 := 0.875\ in$ Diameter bottom bars

$dia_6 := 0.75\ in$ Diameter stirrup bars

$d_{vert} := H_{barrier} - Clr\_cov - dia_6 - 0.5 \cdot dia_7$ $d_{vert} = 61.81\ in$

$$M_{nz} := A_s \cdot f_y \cdot \left(d_{vert} - \frac{a}{2}\right) \cdot \frac{1}{12}$$ $M_{nz} = 1275.3\ kip \cdot ft$

For reinforced concrete sections:

$\phi_f := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored flexural resistance is:

$M_{rz} := \phi_f \cdot M_{nz}$ $M_{rz} = 1147.7 \ kip \cdot ft$

For rectangular section behavior (transverse loading):

$b := H_{barrier}$ $b = 66.00 \ in$

Assume side reinforcement is #6 bars and stirrups are #6 bars:

$A_{st\_6} := 0.44 \ in^2$ $Num\_bars := 8$

$A_s := A_{st\_6} \cdot Num\_bars$ $A_s = 3.52 \ in$

$$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \qquad c = 1.27 \ in$$

$a := \beta_1 \cdot c$ $a = 1.08 \ in$

$Clr\_cov := 2.00 \ in$ Side bar clear cover

$dia_6 := 0.75 \ in$ Diameter of stirrup/side bars

$d_{horiz} := W_{barrier\_str} - Clr\_cov - dia_6 - 0.5 \cdot dia_6$ $d_{horiz} = 35.88 \ in$

$$M_{ny} := A_s \cdot f_y \cdot \left(d_{horiz} - \frac{a}{2}\right) \cdot \frac{1}{12} = 621.934 \qquad M_{ny} = 621.9 \ kip \cdot ft$$

For reinforced concrete sections:

$\phi_f := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored flexural resistance is:

$M_{ry} := \phi_f \cdot M_{ny}$ $M_{ry} = 559.7 \ kip \cdot ft$

If the factored axial load is less than $\phi_c \, f'_c \, A_g$: **LRFD [5.6.4.5]**

$$\frac{M_{uy}}{M_{ry}} + \frac{M_{uz}}{M_{rz}} < 1.00 \qquad \frac{M_{uy}}{M_{ry}} + \frac{M_{uz}}{M_{rz}} = 0.95$$

Is 0.95 < 1.0 ? **Yes** check = OK

E39-1.8 Shear Capacity

For rectangular section behavior (vertical loading):

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n := min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

The nominal shear of the concrete is calculated as follows:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := b$ $b_v = 66.00\ in$

$Clr\_cov := 3.00$ in Bottom bar clear cover

Determine effective shear depth, d_v:

For non-prestressed sections:

$d_e := d_{vert}$ **LRFD 5.7.2.8-2** $d_e = 61.81\ in$

d_v is the maximum of the following three equations: **LRFD 5.7.2.8**

$d_{v1} := d_{vert} - \dfrac{a_{vert}}{2}$ $d_{v1} = 60.73\ in$

$d_{v2} := 0.9 \cdot d_e$ $d_{v2} = 55.63\ in$

$d_{v3} := 0.72 \cdot H_{barrier}$ $d_{v3} = 47.52\ in$

$d_v := \max\left(d_{v1}, d_{v2}, d_{v3}\right)$ $d_v = 60.73\ in$

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $V_c = 473.9\ kips$

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

$\theta := 45$ deg Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 0.88$ in^2 #6 stirrups (2 legs)

$s := 6.00$ in Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s}$$ $V_s = 534.4$ *kips*

$V_{n1} := V_c + V_s + V_p$ $V_{n1} = 1008.3$ *kips*

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $V_{n2} = 3507.0$ *kips*

$V_n := min\left(V_{n1}, V_{n2}\right)$ $V_n = 1008.3$ *kips*

For reinforced concrete sections:

$\phi_v := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$V_{ry} := \phi_v \cdot V_n$ $V_{ry} = 907.5$ *kips*

For rectangular section behavior (transverse loading):

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$V_n := min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$

The nominal shear of the concrete is calculated as follows:

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := b$ $b_v = 66.00$ *in*

$Clr\_cov := 2.00$ *in* Side bar clear cover

Determine effective shear depth, d_v:

For non-prestressed sections:

$d_e := d_{horiz}$ **LRFD 5.7.2.8-2** $d_e = 35.88$ *in*

d_v is the maximum of the following three equations: **LRFD 5.7.2.8**

$d_{v1} := d_{horiz} - \frac{a_{horiz}}{2}$ $d_{v1} = 35.34$ *in*

$d_{v2} := 0.9 \cdot d_e$ $d_{v2} = 32.29$ *in*

$d_{v3} := 0.72 \cdot W_{barrier\_str}$ $d_{v3} = 28.08$ *in*

$d_v := \max(d_{v1}, d_{v2}, d_{v3})$ $\quad d_v = 35.34\ in$

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $\quad V_c = 275.8\ \ kips$

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

$\theta := 45$ $\quad deg$ $\quad$ Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 0.88$ $\quad in^2$ $\quad$ #6 stirrups (2 legs)

$s := 6.00$ $\quad in$ $\quad$ Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s}$$ $\quad V_s = 311.0\ kips$

$V_{n1} := V_c + V_s + V_p$ $\quad V_{n1} = 586.7\ kips$

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $\quad V_{n2} = 2040.7\ \ kips$

$V_n := min(V_{n1}, V_{n2})$ $\quad V_n = 586.7\ kips$

For reinforced concrete sections:

$\phi_v := 0.90$ $\quad$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

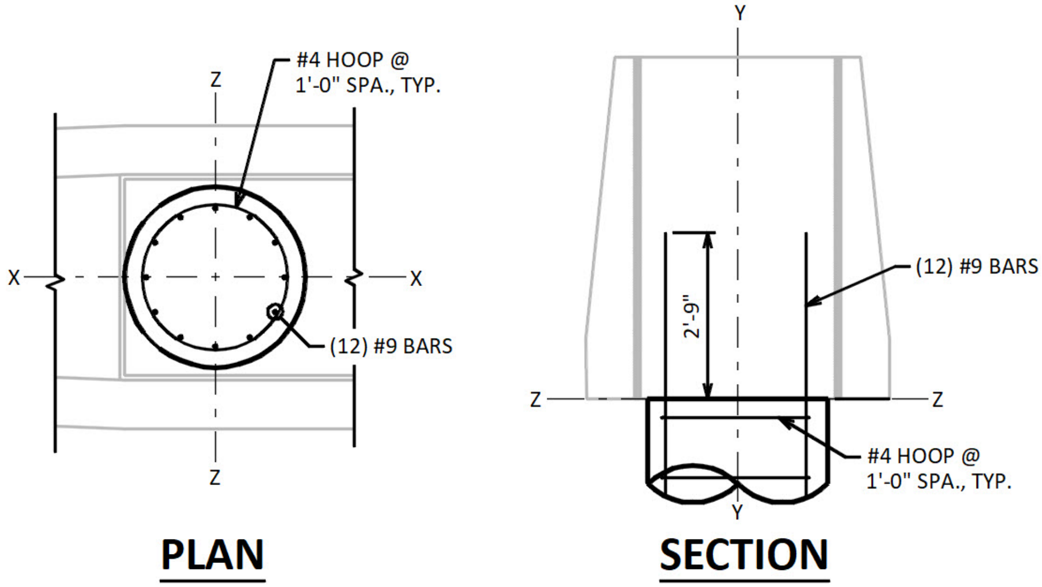
$V_{rz} := \phi_v \cdot V_n$ $\quad V_{rz} = 528.1\ \ kips$

Check combined shear::

$$\frac{V_{uy}}{V_{ry}} + \frac{V_{uz}}{V_{rz}} < 1.0$$

$\frac{V_{uz}}{V_{rz}} + \frac{V_{uz}}{V_{rz}} = 0.47$ $\quad$ Is 0.47 < 1.0 ? **Yes** $\quad$ check = OK

E39-1.9 Check Reinforcement at Top of Drilled Shaft



Check Case II - TL-5 Loading at C/L of drilled shaft, this develops the maximum moment and shear at the top of the drilled shaft. It is assumed that the adjacent pavement prevents the shaft from rotating, therefore only in the interface between the shaft and cap need be check for this loading. This example also conservatively assumes only one shaft resists the TL-5 loading.

$F_x := F_L$ $\qquad F_x = 41.0$ kips

$F_y := 0.5 \cdot W_{DC} \cdot L_{barrier} + F_v$ $\qquad F_y = 108.7$ kips

$F_z := F_t$ $\qquad F_z = 124.0$ kips

$M_x := F_z \cdot H_{barrier} \cdot \frac{1}{12}$ Conservatively ignore vertical load $\qquad M_x = 682.0$ kip•ft

$M_y := F_x \cdot (0.5 \cdot W_{barrier\_top}) \cdot \frac{1}{12}$ $\qquad M_y = 83.7$ kip•ft

$M_z := F_x \cdot H_{barrier} \cdot \frac{1}{12}$ $\qquad M_z = 225.5$ kip•ft

Check shear resistance:

Assume shaft reinforcement is #9 bars vertical with #4 ties:

$dia_4 := 0.50$ in $\qquad dia_9 := 1.128$ in $\qquad clr\_cov := 3.00$ in

The nominal shear resistance of the section is calculated as follows, **LRFD [5.7.3.3]**:

$$V_n := min\left(V_c + V_s + V_p, 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p\right)$$

The nominal shear of the concrete is calculated as follows:

$$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

$\beta := 2$ Simplified procedure **LRFD 5.7.3.4.1**

$\lambda := 1$ Concrete density modification factor **LRFD 5.4.2.8**

$b_v := Diam_{shaft} \cdot 12$ $b_v = 36.00\ in$

$$d_e := \frac{b_v}{2} + \frac{(b_v - 2 \cdot (clr\_cov + dia_4) - dia_9)}{\pi}$$ $d_e = 26.87\ in$

$d_v := 0.9 \cdot d_e$ Effective shear depth **LRFD C5.7.2.8-2** $d_v = 24.18\ in$

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$ $V_c = 102.9\ kips$

The shear resistance provided by transverse reinforcement

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s}$$

$\theta := 45\ deg$ Simplified procedure **LRFD 5.7.3.4.1**

$A_v := 0.40\ in^2$ #4 ties (2 legs)

$s := 12.00\ in$ Stirrup spacing

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot\left(\theta \cdot \frac{\pi}{180}\right)}{s}$$ $V_s = 48.4\ kips$

$V_{n1} := V_c + V_s + V_p$ $V_{n1} = 152.3\ kips$

$V_{n2} := 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p$ $V_{n2} = 762.8\ kips$

$V_n := min(V_{n1}, V_{n2})$ $V_n = 152.3\ kips$

For reinforced concrete sections:

$\phi_v := 0.9$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$V_r := \phi_v \cdot V_n$ $V_r = 137.1\ kips$

$V_u := \sqrt{F_x^2 + F_z^2}$ $V_u = 130.6\ kips$

Is V_u = 130.6 kips < V_r = 142.0 kips? **Yes** check = OK

Check the top of drilled shaft as a reinforced concrete column:

The assessment of the resistance of a compression member with biaxial flexure is dependent upon the magnitude of the factored axial load. If the factored axial load is less than ten percent of the gross concrete strength multiplied by the phi-factor for compression members, then use Equation 5.6.4.5-3. Otherwise, use Equation 5.7.4.5-1. Regardless of which equation in the above paragraph controls, commercially available software is generally used to obtain the moment and axial load resistances.

The procedure as discussed above is carried out as follows:

$\phi := 0.75$ **LRFD [5.5.4.2].** Therefore, the factored shear resistance is:

$$A_g := \frac{\pi \cdot (Diam_{shaft} \cdot 12)^2}{4} \qquad A_g = 1017.9 \quad in^2$$

$$0.10 \cdot \phi \cdot f'_c \cdot A_g = 267.2 \quad kips$$

$$P_r := \phi \cdot f'_c \cdot A_g = 2671.9 \quad kips$$

$$0.10 \cdot P_r = 267.2 \quad kips$$

F_y = 108.7 kips < 267.2 kips Therefore, use **LRFD [Equation 5.6.4.5-3]**

$$M_{ux} := M_x \qquad M_{ux} = 682.0 \quad kip \cdot ft$$

$$M_{uz} := M_z \qquad M_{uz} = 225.5 \quad kip \cdot ft$$

$$M_u := \sqrt{M_{ux}^2 + M_{uz}^2} \qquad M_u = 718.3 \quad kip \cdot ft$$

$$M_r := 719.0 \quad kip \cdot ft$$

$\frac{M_u}{M_r} = 0.999$ Is 0.999 < 1.0 ? **Yes** check = OK

The factored flexural resistances shown above, M_r, was obtained by the use of commercial software.

E39-1.10 Interface Shear Transfer

Check interface shear capacity across construction joint between transition barrier section and foundation cap per **LRFD 5.7.4**.

Per SDD-14B32 the standard barrier transition section has 6 - #5 horizontal bars on each face continuing across the interface construction joint between the barrier transition and foundation cap sections.

Area of shear reinforcement crossing the shear plane

$A_{st\_5} := 0.31 \; in^2$ Area of #5 bar

$A_{vf} := 2 \cdot 6 \cdot A_{st\_5}$ $A_{vf} = 3.72 \; in^2$

Calculate shear resistance. For purpose of determining shear transfer contact area, use gross combined area of resisting foundation cap section and integral barriers.

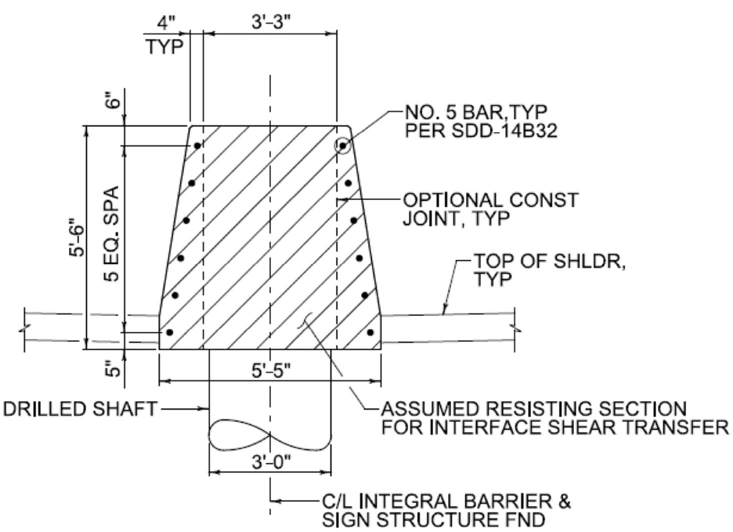
$A_{cv} := Area_{barrier} \cdot 144$ $A_{cv} = 3671 \quad in^2$

Check that minimum shear interface reinforcement is provided per LRFD 5.7.4.2:

$$A_{vf\_min} := \frac{0.05 \cdot A_{cv}}{f_y}$$ **LRFD 5.7.4.2-1** $A_{vf\_min} = 3.06 \quad in^2$

Is $A_{vf\_min} <$ A_{vf} = 3.72 in^2 **Yes** check = OK

Summary: Due to the 1" filler between the cap and barrier shown in SDD-14B32, there is no friction between the concrete surfaces, shear is resisted by reinforcing steel only. Shear interface reinforcement of 12 - #5 bars per SDD-14B32 is adequate.



INTEGRAL BARRIER SECTION

Check interface shear capacity across construction joint between drilled shaft and foundation cap per **LRFD 5.7.4**. Conservatively assumes a single shaft.

(12) #9 bars cross the shear plane at the top of the drilled shaft.

$$A_{st\_9} := 12 \cdot \pi \cdot \left(\frac{dia_9}{2}\right)^2$$ $A_{st\_9} = 11.992 \; in^2$

Calculate interface shear resistance. For purpose of determining shear transfer contact area, use gross combined area of resisting foundation cap section and one drilled shaft.

$A_{cv\_shaft} := \pi \cdot \left(\frac{Diam_{shaft}}{2}\right)^2 \cdot 144$ $A_{cv\_shaft} = 1017.876$ in^2

Check that minimum shear interface reinforcement is provided per LRFD 5.7.4.2:

$A_{vf\_min} := \frac{0.05 \cdot A_{cv\_shaft}}{f_y}$ **LRFD 5.7.4.2-1** $A_{vf\_min} = 0.85$ in^2

Is $A_{vf\_min} < A_{st\_9} = 11.992\, in^2$ **Yes** check = OK

Calculate factored interface shear force due to TL-5 vehicular collision forces only:

$V_{CT} := \left(V_{z\_CT}{}^2 + V_{y\_CT}{}^2\right)^{0.5}$ $V_{CT} = 147.57$ *Kips*

Vehicle collision force is extreme event limit state, therefore load factor = 1.0:

$V_{ui} := 1.0 \cdot V_{CT}$ $V_{ui} = 147.57$ *Kips*

Assume clean construction joint, not intentionally roughened. Per **LRFD 5.7.4.3**:

$c_{cv} := 0.075$

$\mu := 0.6$

$K_1 := 0.2$

$K_2 := 0.8$

Permanent axial compression across shear interface = 0 (conservative)

The nominal shear interface (shear friction) capacity is the smallest of following three equations:

$V_{nsf1} := c_{cv} \cdot A_{cv\_shaft} + \mu \cdot A_{st\_9} \cdot f_y$ **LRFD 5.7.4.3-3** $V_{nsf1} = 508.05$ *Kips*

$V_{nsf2} := K_1 \cdot f'_c \cdot A_{cv\_shaft}$ **LRFD 5.7.4.3-4** $V_{nsf2} = 712.51$ *Kips*

$V_{nsf3} := K_2 \cdot A_{cv\_shaft}$ **LRFD 5.7.4.3-5** $V_{nsf3} = 814.3$ *Kips*

Nominal shear interface (shear friction) capacity:

$V_{nsf} := min\left(V_{nsf1}, V_{nsf2}, V_{nsf3}\right)$ $V_{nsf} = 508.05$ *Kips*

Factored shear interface resistance; for extreme event loading:

$\phi_{si} := 1.0$ **LRFD 5.7.4.3**

Therefore, the factored interface shear resistance is:

$V_{ri} := \phi_{si} \cdot V_{nsf}$ $V_{ri} = 508.05$ *Kips*

$\frac{V_{ui}}{V_{ri}} = 0.2905$ Is V_{ui} < V_{ri} = 508.25 kips? **Yes** check = OK

Summary: Shear interface reinforcement at the top of the shaft of 12 - #9 bars is adequate. A shear key should be provided at the top of the shaft as shown in the standard plans.

Table of Contents

40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

removed to at least the original deck surface. Additional surface milling may not be practical if the previous overlay included a milling operation.

40.5.1.3 Polyester Polymer Concrete Overlay

A polyester polymer concrete (PPC) is expected to extend the service life of a bridge deck for 20 to 30 years. This system is a mixture of aggregate, polyester polymer resin, and initiator; which can be placed as a deck overlay using conventional concrete mixing and placement equipment, albeit most likely dedicated to PPC usage. The main advantages of a PPC overlay is that it is impermeable and causes minimal traffic disruptions due to its quick cure time. High costs and lack of performance data are the main disadvantages.

Prior to the placement of the PPC overlay, a high molecular weight methacrylate (HMWM) binder is placed on the prepared deck. This bonds the overlay to the deck, and it also serves to seal existing cracks in the deck. When the existing concrete is in good condition, PPC is effective at mitigating chloride penetration due to its impermeability. In some situations, PPC has exhibited reflective cracking from the deck below. Cracks should be sealed with methacrylate sealer as recommended by the particular PPC manufacturer.

The total thickness of a PPC overlay is typically 3/4” to 1”. While thicker overlays are possible, they are usually cost prohibitive. PPC can be placed at 3/4” thick as opposed to a typical 1 1/2” thick concrete overlay. This may help in situations where bridge ratings and/or profile adjustments are of concern but should not be the primary reason for applying PPC.

Since most applications recommend a 1-inch or less overlay, PPC overlays are considered a thin polymer overlay and have similar requirements and restrictions. PPC overlays should be limited to decks in good condition that require shorter traffic disruptions for sites with high traffic volumes and lane closure restrictions. PPC is a durable product and has a relatively fast curing time (2 to 4 hours), but also has a higher cost as compared to a concrete overlay. PPC overlays should be used based on the following restrictions:

- Deck wearing surface distress should not exceed 5% of the total deck area.

- Decks should have a NBI rating of 6 or greater and be less than 20 years old. Older decks may be considered when the existing deck has been protected by a thin polymer overlay or when chloride testing indicates acceptable chloride levels at the reinforcement. Chloride contents at the reinforcement should not exceed 2 lbs/CY for decks with epoxy coated reinforcement. PPC overlays are not recommended on decks with uncoated top mat reinforcement. Decks exposed to chlorides, exceeding 10 years, should consider a ¾-inch minimum scarification to remove chlorides.

- PPC overlays should not be placed on concrete decks or Portland cement concrete patches less than 28 days, unless approved otherwise. Patch and crack repairs shall be compatible with the overlay material.

- PPC shall not be used for structural repairs due to costs and performance concerns.

- PPC should not be used unless lower cost preservation treatments (e.g. thin polymer overlays) have proven ineffective. If a bridge deck has a TPO, chloride ion testing will be performed near the end of the TPO life to determine eligibility of either a TPO reapplication or a PPC overlay (if the structure also meets the condition and ADT criteria). If the average chloride concentration at 1” depth is less than 1 lb/cy, the existing TPO is considered an effective preservation treatment. The TPO should be replaced with anther TPO. If the average chloride concentration at 1” depth is greater than 1 lb/cy, a PPC overlay should be considered (including a ¾”-1” milling of the deck surface to remove chlorides).

Note: PPC overlays are expensive and new to WisDOT. As a result, use of PPC overlays should be limited to preservation projects that meet the requirements outlined in Figure 40.5-2 **or as approved by the Bureau of Structures.**

Other factors which may affect BOS approval of PPC overlays include:

- Proximity to other high ADT roadways (i.e., service ramp)
- Backbone/Interstate or high priority structure
- Preservation of slab structure
- Presence of active cracking (not recommended when active cracking is present)
- Enhanced friction (not to be used as a ride correction or as a high-friction improvement)

40.5.1.4 Polymer Modified Asphaltic Overlay

A polymer modified asphaltic (PMA) overlay is expected to extend the service life of a bridge deck for 10 to 15 years. This system is a mixture of aggregate, asphalt content, and a thermoplastic polymer modifier additive, which can easily be placed as a deck overlay using conventional asphalt paving equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

The added polymer allows for the overlay to resist water and chloride infiltration. Proper mix control and placement procedures are critical in achieving this protection. Core tests have shown the permeability of this product is dependent on the aggregate. As a result, limestone aggregates should not be used.

PMA overlays can be used on more flexible structures (e.g. timber decks or timber slabs) and to minimize traffic disruptions.

Designers should contact the region to determine if a PMA overlay is a viable solution for the project. In some areas, product availability or maintaining an acceptable temperature may be problematic.

Note: PMA overlays are expensive, have a limited service life relative other overlay types, and product availability may be problematic. As a result, PMA overlays usage should be limited.

40.5.1.5 Asphaltic Overlay

An asphaltic overlay, without a waterproofing membrane, is expected to extend the service life of a bridge deck for 3 to 7 years. This system may be a viable treatment if the deck or bridge is programmed for replacement within 4 years on lightly traveled roadways and is able to provide a smooth riding surface. Without a waterproofing material, the overlay may trap moisture at the existing deck surface, which may accelerate deck deterioration.

These overlays must be watched closely for distress as the existing deck surface problems are concealed. This system is typically an asphaltic pavement with a mixture of aggregates and asphaltic materials, which can easily be placed as a deck overlay using conventional asphaltic mixing and placement equipment. The thickness of the overlay is 2-inches minimum and can accommodate profile and cross-slope differences.

Note: Asphaltic overlays, without a waterproofing membrane, are not eligible for federal funds.

40.5.1.6 Asphaltic Overlay with Waterproofing Membrane

An asphaltic overlay, with a waterproofing membrane, is currently being used on a very limited basis. This system is expected to extend the service life of a bridge deck for 5 to 15 years. Experience indicates that waterproofing membranes decrease the rate of deck deterioration by preventing or slowing the migration of water and chloride ions into the concrete.

In the 1990's, waterproofing membranes were actively used with asphaltic overlays for protecting existing decks, but were phased out by 2009 when they were restricted due to performance concerns and the inability to inspect the deck. As a result, low slump concrete or PMA overlays are currently recommended when deck or bridge replacements are programed beyond 4 years, unless approved otherwise.

Note: Asphaltic overlays, with a waterproofing membrane, requires prior-approval by the Bureau of Structures. This system is currently under review for possible improvements.

40.5.1.7 Other Overlays

Several other overlay systems have been used on past projects, but are generally not used currently. Use of these systems or other systems not previously mentioned require prior-approval by the Bureau of Structures.

- Micro-silica (silica-fume) modified concrete overlay – Provides good resistance to chloride penetration due to its low permeability.

- Latex modified concrete overlay – Provides a long-lasting overlay system with minimal traffic disruptions. Several other states are currently using this overlay method with hydrodemolition deck preparations.

- Reinforced concrete overlays:

    - Thin overlays (< 4 ½”) – Uses a superplasticizer and fiber reinforcement (steel or synthetic) for additional crack control by reducing cracks and crack widths.

    - Thick overlays (≥ 4 ½”) – Uses steel reinforcements, rebar or weld wire fabric, typically for new structural decks. This overlay is intended to provide at least one layer of steel reinforcement, in each direction, for crack control. This overlay is currently recommended for PS box girder superstructures, which allows for composite details and improved means to control longitudinal reflective cracking. For most cases, steel reinforcement is not required when rehabilitation overlays exceed 4 1/2 - inches. Use of low slump Grade E concrete may not be suitable when incorporating steel reinforcements.

40.5.2 Selection Considerations

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

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

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

Table 40.5-1
Overlay Selection Considerations

Overlay Type	Advantages	Disadvantages	Notes
Thin Polymer Overlay	• Minimal dead load • Minimal traffic disruptions • Seals the deck • Provides traction	• Requires a concrete age of at least 21 days • Requires decks with minimal defects and low chloride concentrations • Sensitive to moisture, temperature, and humidity at placement • Reflective cracking resistance concerns	
Low Slump Concrete Overlay	• Contractor familiarity and department experience • Long life span potential • Durable • Ease to accommodate grade differences and deficiencies	• Traffic disruptions • Additional dead load • High maintenance requirements • Railing height concerns • Susceptible to cracking • Specialized finishing equipment	• May require crack sealing the following year and periodically thereafter.
Polyester Polymer Concrete Overlay	• Minimal dead load • Minimal traffic disruptions • Seals the deck • Provides traction • Long life span potential • Durable • Low maintenance requirements	• High cost • Dedicated equipment • Limited usage in Wisconsin • Sensitive to moisture, temperature, and humidity at placement	• Requires BOS Prior-Approval
Polymer Modified Asphaltic Overlay	• Minimal traffic disruptions • Ease to construct • Can be used on more flexible structures (e.g. timber decks or timber slabs)	• High cost • Susceptible to permeability • Difficult to assess top of deck condition	• Contact region for availability • Minimal research has been performed on the durability of this system in Wisconsin
Asphaltic Overlay	• Low cost • Ease to construct • Ease to accommodate grade differences and deficiencies	• Short life span • Not eligible for federal funds • Overlay permeability • Difficult to assess top of deck condition	• Deck or bridge replacement should be programmed within 4 years
Asphaltic Overlay with Membrane	• Ease to construct • Minimal traffic disruptions • Long life span potential • Can be used on more flexible structures (e.g. PS box girders)	• Susceptible to permeability • Requires a membrane • Difficult to assess top of deck condition	• Currently under review • Requires BOS Prior-Approval

Table 40.5-2

Overlay Advantages, Disadvantages, and Notes

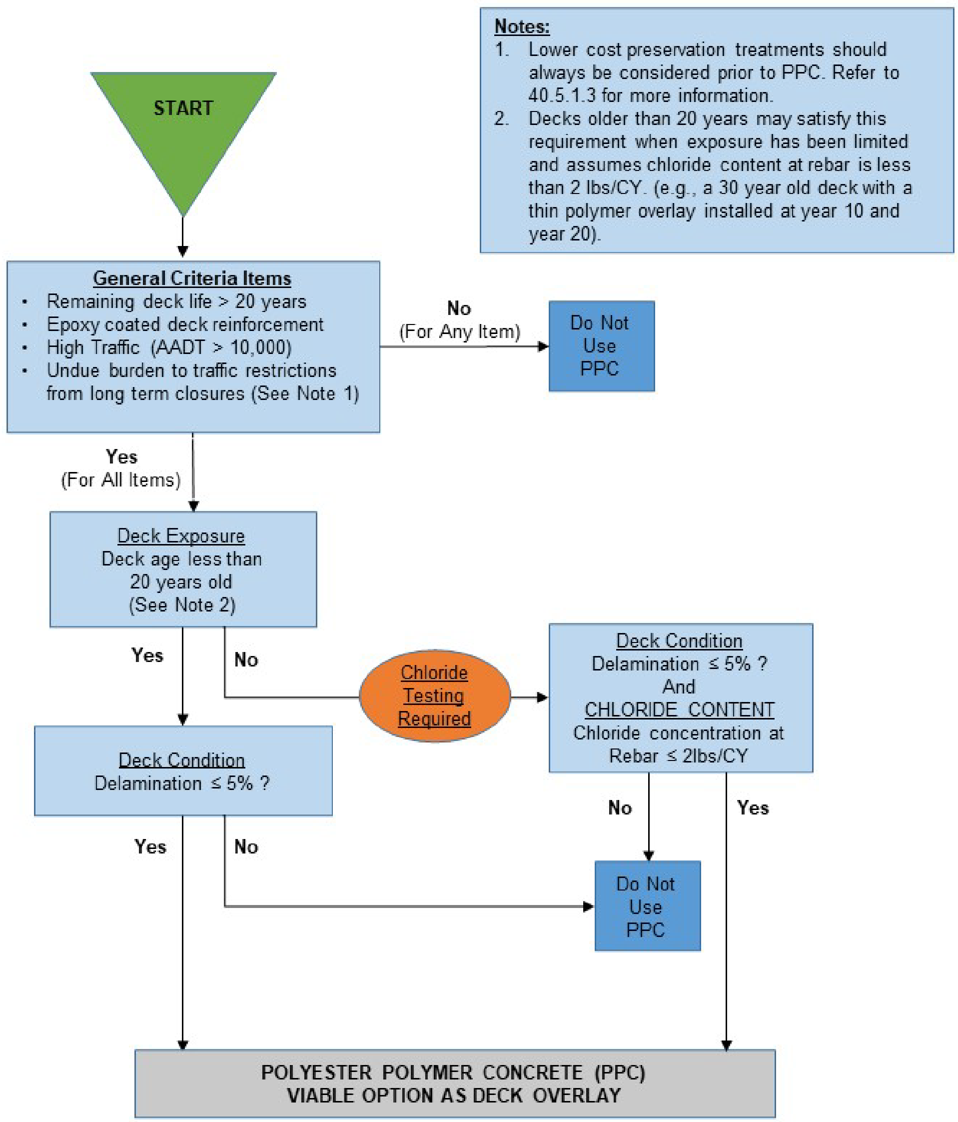


Figure 40.5-2

Polyester Polymer Concrete Overlay Usage Flowchart

40.5.3 Deck Assessment

The following are common deck assessment tools that can be used to survey existing deck conditions:

- Visual Inspections – Used to detect surface cracks, discontinuities, corrosion, and contamination.

- Audible Inspections - The two most common types of audible inspections are chain dragging and hammer sounding. Chain dragging is normally used on large concrete surface areas, such as bridge decks, while hammer sounding can be used on a number of materials in random locations. Both methods typically rely on the experience of the inspector to differentiate the relative sounds of similar materials.

- Infrared thermography - Infrared Thermography (IR) is an alternative tool for locating and mapping delaminations in bridge decks and pavements. A technique using an infrared scanner and control video camera, infrared thermography senses temperature differences between delaminated and non-delaminated areas.

- Ground penetrating radar (GPR) – GPR is a technique using electromagnetic signals, which can detect dielectric differences. This method can be used to measure concrete cover, overlay thickness, and reinforcing steel locations. This method can also be used to locate delaminations.

- Deck cores – Cores can be used to determine existing overly thicknesses, concrete cover, and concrete strength. As-built plans should only be used as a reference for existing conditions. Additionally, cores can be used to determine chloride content profiles. For asphaltic overlay, coring may be the best tool for deck assessments.

- Chloride Ion Testing - Chloride ions are the major cause of reinforcing steel corrosion in concrete. In evaluating chloride content, it is recommended that a chloride profile (chloride concentration percentage versus depth measurement below the concrete surface) be developed. This profile is important for assessing the future corrosion susceptibility of steel reinforcing and in determining the primary source of chlorides.

- Half-cell potential testing - A method used to detect whether the reinforcing steel is under active corrosion.

Visual inspections, audible inspections, and IR are the most common deck assessment tools for identifying delaminations and unsound concrete. For more information on deck assessment tools, refer to the Structure Inspection Manual – Part 5 – NDE and PDE Testing. Deck condition surveys should be placed on the structures plans. This should include the survey type and date when the survey was completed.

40.5.4 Deck Preparations

Prior to placing overlays, the existing deck surface will require deck preparations to repair the existing deck and to ensure that the overlay is properly bonded to the existing concrete. These preparations can range from sand blasting the entire deck to milling the entire deck with extensive repairs and are dependent on the existing deck conditions (distress, chloride concentration, existing overlay, proposed overlay, etc.).

The below deck preparations are typically used prior to placing overlays. Check the latest specifications for additional information.

Concrete Removal

Concrete deck removal usually includes the removal of unsound surface materials and the removal of a predetermined depth to remove concrete with high chloride concentrations. The following techniques can be used for large concrete removal areas:

> Mechanical scarification or milling – The removal of existing deck to predetermined depth using a milling machine and other approved operations. This process can remove concrete with high chloride contents. However, this aggressive removal process has the potential to introduce micro-cracking into the existing deck.
>
> Hydrodemolition – The removal of existing deck to a predetermined depth and the ability to selectively remove distressed areas using ultra high-pressure water-jetting (above 25,000 psi). A benefit to this process is that it does not introduce micro-cracking. WisDOT has very limited experience with this process and is usually cost prohibitive.

Generally, decks receiving a low slump concrete overlay will also include a 1-inch minimum deck removal. This assumes the existing top of deck has been exposed long enough to develop high chloride concentrations and would benefit from a milling operation. For early aged or protected (e.g. polymer overlay) decks, concrete milling may not be necessary prior to the overlay application and may be deferred to future overlay applications. Typically, only one aggressive milling operation is practical for a deck to leave sufficient cover for future overlays. Maintain ½” to 1” of rebar cover to ensure proper bonding and to protect the rebar and coating during the milling operation.

Deck Repairs

Care should be taken to limit damaging sound concrete and the existing reinforcement. Use of appropriate tools, hammers no more than 35 pounds and no more than 15 pounds when within one inch of the steel, is intended to limit distressed areas and avoid full-depth repairs. Additionally, saw cut depths should be carefully monitored such that the existing steel is not cut.

Cathodic protection may be warranted for decks with a high chloride content to help prevent corrosion from initiating.

The following items are associated with repairing distressed deck areas as shown in Figure 40.5-3:

Preparation Decks Type 1 – The removal of existing patches and unsound concrete only to a depth that exposes 1/2 of the peripheral area of the top or bottom bar steel in the top mat of reinforcement. Care should be taken to limit damaging sound concrete.

Preparation Decks Type 2 – The removal of existing unsound concrete below the limit of the type 1 removal described above. One inch below the bottom of the top or bottom bar steel in the top mat of reinforcement is the minimum depth of type 2 removal.

Full-Depth Deck Repair – The complete removal of existing concrete.

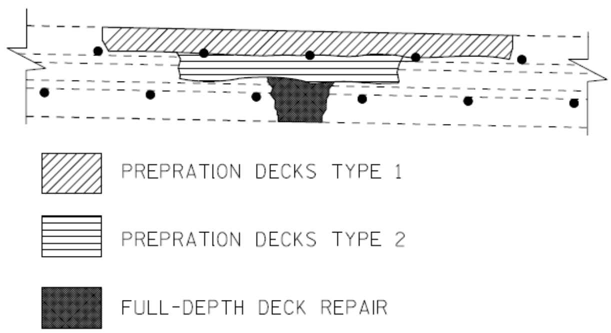


Figure 40.5-3
Deck Repairs

Deck Patches

Portland cement concrete is the preferred patch material. This material is easy to work with and very economical. When traffic impacts warrants, other materials may be considered. For concrete overlays, Type 1 and Type 2 deck patch repairs should be filled during the concrete overlay placement. Full-depth deck repairs should not be filled during the concrete overlay placement, but rather filled and curing a minimum of 24 hours before placing the concrete overlay. For other overlays, concrete repairs are usually properly cured prior to placing the overlay.

For minimal traffic impacts, a rapid-set material may be used for deck patches on asphaltic and thin polymer overlays. When repair quantities are minimal, distress areas less than 5% of the entire deck area, PPC overlays may use PPC to fill deck repairs prior overlay placement. See Table 40.5-1 for typical deck patch materials. Refer to the approved products list for a list of pre-qualified rapid setting concrete patch materials and their associated restrictions.

Surface Removal and Surface Preparation

Overlays require a properly prepared deck to achieve the desired bond strength. The following techniques are used for deck surface removal and preparations for an overlay:

Air cleaning – A preparation process to remove loose materials with compressed air. This process is intended to remove any material that may have gathered after the use of surface or concrete removal processes. This process is performed just prior to installing the overlay.

Water blasting (pressure or power washing) - A preparation process used to remove loose materials using low to high pressure water (5,000 psi to 10,000 psi). This process is beneficial as it keeps down dust and can remove loose particles.

Sand blasting – A surface removal process to remove loose material, foreign material, and loose concrete with sand material.

Shot blasting – A surface removal process to remove loose material, foreign material, and loose concrete by propelling steel shot against the concrete surface. This process also provides a roughen surface texture for improved bonding for overlays. Note: TPO's and PPC overlays provisions required a concrete surface profile meeting CSP-5 prior to overlay placement. This surface profile can be achieved using medium to medium-heavy shot blast.

40.5.5 Preservation Techniques

The following are some of the common activities being used to preserve decks and overlays:

- Deck cleaning (sweeping and power washing)
- Deck sealing/crack sealing
- Joint cleaning
- Joint repairs
- Deck patching

For additional preservation techniques and information refer to Chapter 42-Bridge Preservation.

40.5.5.1 Deck Sealing

Deck sealing has been found to be a cost-effective tool in preserving decks and overlays. In general, deck treatments should be applied as early as possible and re-applied thereafter. The frequency of deck sealing is dependent on the roadway traffic volume. Decks are to be sealed at initial construction and then resealed at the frequency shown in Table 40.5-3**.** Decks are to be resealed twice prior to applying a thin polymer overlay. Crack sealing should be considered as a potential combined treatment when deck sealing.

Roadway ADT	Deck Sealing Frequency
ADT < 2,500	4 – 5 years
2,500 <= ADT < 6,500	4 years
6,500 <= ADT < 15,000	3 years
ADT >= 15,000	2 years*

*In place of deck sealing, a thin polymer overlay is recommended within 2 years of deck construction. Use of the thin polymer overlay at this time will help minimize traffic impacts related to deck preservation work.

Table 40.5-3
Deck Sealing Frequency

Thin polymer overlays can be used in lieu of resealing the deck on a project-to-project basis with BOS approval. Approval occurs through the structure certification process. Some examples where TPOs might be used instead of deck sealing are where heavy snowmobile traffic is expected or when the safety certification provides justification for enhanced friction surface treatment.

40.5.6 Other Considerations

- Bridges with Inventory Ratings less than HS10 after rehabilitation shall not be considered for overlays, unless approved by the Bureau of Structures Design Section.

- Inventory and Operating Ratings shall be provided on the bridge rehabilitation plans.

- Verify the desired transverse cross slope with the Regions as they may want to use current standards.

- On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans. If more than 1/3 of the steel is exposed and the bar ends are not anchored, either adjacent spans must be shored or a special analysis and removal plan are required. Reinforcement shall be anchored using Portland cement concrete.

- Asphaltic overlays should not be considered on a bridge deck which has a longitudinal grade in excess of four percent or an extensive amount of stopping and starting traffic.

- All full-depth repairs shall be made with Portland cement concrete.

- Joints and floor drains should be modified to accommodate the overlay

- Concrete chloride thresholds – Chloride content tests measure the chloride ion concentrations at various depths. Generally, research has shown initiation of corrosion is expected when the chloride content is between 1 to 2 lbs/CY in concrete for uncoated bars and 7 to 12 lbs /CY for epoxy coated bars at the reinforcement. These limits are referred to as the threshold for corrosion. Threshold limits do not apply to stainless steel rebar.

When the chloride ion content is greater than 0.8 lbs/CY in concrete for uncoated bars and 5 lbs /CY for epoxy coated bars at the reinforcement depth, measures should be considered to limit additional chloride infiltration.

- See Chapter 6-Plan Preparation and Chapter 40 Standards for additional guidance.

- Refer the standard details for the most current bid items.

- Overlay transitional areas should be used and coordinated when accommodating profile differences. These transitions are intended to improve ride quality and protect against snowplow damage. Ideally, transitions are placed such that the overlay thickness remains constant, which requires a tapered removal of the existing surface over a sufficient distance. For profile adjustments 1 1/2-inch or greater, transitional areas should consider a minimum taper rate of 1:250 for low-speed applications (RSD< 50 mph) and for high-speed applications up to a 1:400 taper rate. Typically, thicker profile adjustments are provided off the bridge deck and are coordinated by the roadway designer. For profile adjustments less than 1 1/2-inch, a minimum rate of 1:250 may be used regardless of the roadway design speed. For a 3/4-inch minimum PPC overlay, provide a 16-feet minimum transition length. For a 1/4-inch TPO overlay, a 3-feet minimum transition length is sufficient. See Chapter 40 Standards for additional guidance.

40.5.7 Past Bridge Deck Protective Systems

In the past, several bridge deck protective systems have been employed on the original bridge deck or while rehabilitating the existing deck as described in 17.8. The following systems have been used to protect bridge decks:

- Epoxy coated deck reinforcement – Prior to the 1980's, uncoated (black) bars were used throughout structures, including bridge decks. Criteria for epoxy coated reinforcement was first introduced in 1981 as a deck protective system. At first, usage was limited to the top mat of deck reinforcement. By 1987, coated bars were required in the top and bottom mats for high volume roadways (ADT > 5000). By 1991, coated bars were required for all State bridges and on some local bridges (ADT > 1000). Currently, use of epoxy coated deck reinforcement is required on all bridge decks.

- Asphaltic overlay with Membranes – Use of this overlay system was largely discontinued in the 1990's.

- High Performance Concrete (HPC) - Use of HPC has been limited to Mega Projects.

- Thin Polymer Overlays – Use of this overlay system is currently being used.

- Polyester Polymer Concrete Overlays – Use of this overlay system currently being used limitedly.

- Additional Concrete Cover – Use of additional clear cover (> 2 ½ inches) has been used on bridges with high volume and high truck traffic.

- Stainless steel deck reinforcement – Use of stainless steel has been very limited.

- Fiber reinforce polymer (FRP) deck reinforcement - Use of FRP reinforcement has only be used for experimental purposes.

As-built plans should be reviewed for past deck protective systems to assist with the appropriate rehabilitation measures.

40.5.8 Railings and Parapets

Overlays may decrease the parapet height when the existing overlay is not milled off and replaced in-kind. See Chapter 30-Railings for guidance pertaining to railings and parapets associated with rehabilitation structures projects.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges (does not include local roadways over STN routes) eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:

Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the FDM and FDM SDD 14b7 for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, replace existing intermediate concrete diaphragms with new steel diaphragms at existing diaphragm locations (i.e. don't add intermediate lines of diaphragms). See Chapter 19 Standard Details and Steel Diaphragm Insert Sheets for additional information. Existing concrete diaphragms, in good condition, that are full-depth to the bottom of the girder (typically located at the abutments and piers) shall not be removed for a deck replacement.

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.

For staged deck replacement projects, temporary overhangs supporting traffic operations shall be evaluated by the designer. When temporary support is determined necessary (i.e. when the existing temporary exterior girder is unable to reasonably support the temporary overhang condition), the designer should consider either reducing the overhang by modifying the traffic operations or provide a Temporary Support SPV bid item. Note: the bid item Temporary Support is intended to be used when the designer has determined there is a viable path forward through a temporary support system and is not intended to be used for typical overhang falsework stabilization, which is covered in Section 502.3.2.3 of the *Standard Specifications* and incidental to the concrete bid item. Nonetheless, the contractor is generally responsible to provide temporary support during construction.

40.7 Rehabilitation Girder Sections

WisDOT BOS has retired several girder shapes from standard use on new structures. The 36", 45", 54" and 70" girder sections shall be used primarily for bridge rehabilitation projects such as girder replacements or widening.

These sections may also be used on a limited basis on new curved structures when the overhang requirements cannot be met with the wide-flange girder sections. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. See the Standard Details for the girder sections' draped and undraped strand patterns.

The 36", 45", 54", and 70" girders in Chapter 40-Bridge Rehabilitation standards have been updated to include the proper non-prestressed reinforcement so that these sections are LRFD compliant. Table 40.7-1 provides span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder rehabilitation sections. Girder spacing and span lengths are based on the following criteria:

- Interior girders with low relaxation strands at $0.75f_{pu}$,
- A concrete haunch of 2-1/2",
- Slab thicknesses from Chapter 17-Superstructure - General,
- A future wearing surface of 20 psf,
- A line load of 0.300 klf is applied to the girder to account for superimposed dead loads,
- 0.5" or 0.6" dia. strands (in accordance with the Standard Details),
- f'_c girder = 8,000 psi,
- f'_c slab = 4,000 psi, and
- Required f'_c girder at initial prestress < 6,800 psi

36" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	76	82
6'-6"	74	80
7'-0"	69	78
7'-6"	66	76
8'-0"	65	75
8'-6"	63	69
9'-0"	62	67
9'-6"	60	65
10'-0"	59	64
10'-6"	58	63
11'-0"	51	61
11'-6"	50	60
12'-0"	49	58

45" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	102	112
6'-6"	100	110
7'-0"	98	108
7'-6"	96	102
8'-0"	94	100
8'-6"	88	98
9'-0"	88	96
9'-6"	84	90
10'-0"	84	88
10'-6"	82	86
11'-0"	78	85
11'-6"	76	84
12'-0"	70	80

54" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	130	138
6'-6"	128	134
7'-0"	124	132
7'-6"	122	130
8'-0"	120	128
8'-6"	116	124
9'-0"	112	122
9'-6"	110	118
10'-0"	108	116
10'-6"	106	112
11'-0"	102	110
11'-6"	100	108
12'-0"	98	104

70" Girder		
Girder Spacing	Single Span	2 Equal Spans
6'-0"	150*	160*
6'-6"	146*	156*
7'-0"	144*	152*
7'-6"	140*	150*
8'-0"	138*	146*
8'-6"	134*	142*
9'-0"	132*	140*
9'-6"	128*	136
10'-0"	126*	134
10'-6"	122	132
11'-0"	118	128
11'-6"	116	126
12'-0"	114	122

Table 40.7-1
Maximum Span Length vs. Girder Spacing

*For lateral stability during lifting these girder lengths will require pick up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.

Table of Contents

must be taken into account). Appropriate consideration of the haunch is the responsibility of the load rating engineer.

45.6.1.1.1 Variable Girder Spacing (Flare)

Girder spacings may vary over the length of a given girder (flared girder configuration). Some analysis software may allow for a varying distribution factor along the length of the girder. This is the most accurate and thus preferred method for dealing with a flared girder layout.

Alternatively, conservative assumptions may be made regarding the live load distribution and the assigned dead load to the girder being analyzed. The rating engineer is responsible for determining the appropriate assumptions and for ensuring that they produce conservative results. The methods described in **LRFD [C4.6.2.2.1]** are acceptable. All assumptions made shall be clearly noted in the calculations and in the load rating summary sheet (See 45.9.1).

45.6.1.2 Box and Channel Girders

For adjacent prestressed box and channel girders, the concrete topping may be considered structural when rebar extends from the girders up into the concrete topping.

45.6.2 Cast-in-Place Concrete

45.6.2.1 Slab (Flat or Haunched)

WisDOT exception to AASHTO:

When using Load Factor Rating (LFR) and calculating the single lane load distribution factor for concrete slab bridges, the wheel load distribution width, E, shall be taken as 1.71 (12.0 ft/7.0 ft) times the multi-lane distribution width. This conversion is an exception to the AASHTO Standard Specification, which does not indicate the effective slab width for single-lane loading.

Some concrete slab bridges may have been designed with an integral concrete pier cap. This would take the form of increased transverse reinforcement at the pier, most likely combined with a haunched slab design. It is WisDOT experience that the integral pier cap will very rarely control the load ratings and a specific evaluation is not required. However, if the pier cap shows signs of distress, a more detailed load rating evaluation may be required. Consult the Bureau of Structures Load Rating Unit in these cases.

45.6.3 Steel

Consistent with the WisDOT policy item in 24.6.10, moment redistribution should not be considered as a part of the typical rating procedure for a steel superstructure. Moment redistribution may be considered for special cases (to avoid a load posting, etc.). Contact the Bureau of Structures Rating Unit with any questions on the use of moment redistribution.

Plastic analysis shall be used for steel members as permitted by AASHTO specifications, including (but not limited to) Article 6.12.2 (LRFR) and Articles 10.48.1, 10.53.1.1, and 10.54.2.1 (LFR). Plastic analysis shall not be used for members with significant deterioration.

Per code, sections must be properly braced in order to consider plastic capacity. For questions on the use of plastic analysis, contact the Bureau of Structures Rating Unit.

If there are no plans for a bridge with a steel superstructure carrying a concrete deck, it shall be assumed to be non-composite for purposes of load rating unless there is sufficient documentation to prove that it was designed for composite action and that shear studs or angles were used in the construction.

When performing a rating on a bridge with a steel superstructure element (deck girder, floorbeam, or stringer) carrying a concrete deck, the element should be assumed to have full composite action if it was designed for composite action and it has shear studs or angles that are spaced at no more than 2'-0" centers.

Steel girder bridges in Wisconsin have not typically been designed to use the concrete deck as part of a composite system for negative moment. A typical design will show a lack of composite action in the negative moment regions (i.e., no shear studs). However, if design drawings indicate that the concrete deck is composite with the steel girder in negative moment regions, the negative moment steel in the concrete deck shall conservatively consist of only the top mat of steel over the piers.

For steel superstructures, an additional dead load allowance should be made to account for miscellaneous items such as welds, bolts, connection plates, etc., unless these items are all specifically accounted for in the analysis. See 24.4.1.1 for guidance on this additional dead load allowance. Alternatively, the actual weight of all the miscellaneous items can be tabulated and added to the applied dead load.

WisDOT policy items:

When load rating in-service bridges, WisDOT does not consider the overload limitations of Section 10.57 of the AASHTO Standard Specification.

45.6.3.1 Fatigue

For structures originally designed using LRFD, fatigue shall not be part of the rating evaluation.

For structures originally designed using ASD or LFD, fatigue ratings shall not be reported as the controlling rating. However, a fatigue evaluation may be considered for load ratings accompanying a major rehabilitation effort, if fatigue-prone details (category C or lower) are present. Fatigue detail categories are provided in **LRFD Table [6.6.1.2.3-1]**. Contact WisDOT Bureau of Structures Rating Unit on appropriate level of effort for any fatigue evaluation.

45.6.3.2 Rolled I-Girder, Plate Girder, and Box Girder

Application of the lever rule in calculating distribution factors for exterior girders may be overly conservative in some short-span steel bridges with closely spaced girders and slab overhangs. In this case, the live load bending moment for the exterior girder may be determined by applying the fraction of a wheel line determined by multiplying the value of the interior stringers or beams by:

W_e/S, where:

W_e = Top slab width as measured from the outside face of the slab to the midpoint between the exterior and interior stringer or beam. The cantilever dimension of any slab extending beyond the exterior girder shall not exceed S/2, measured from the centerline of the exterior beam.

S = Average stringer spacing in feet.

Alternately, live load distribution for this case may be determined by refined methods of analysis or with consideration of lane stripe placement as described in 45.5.1.2.

It is common practice to use the average haunch height in order to locate the concrete deck in relation to the top of the girder. It is also acceptable to use the actual, precise haunch thicknesses, if they are known. Absent information on the depth of the haunch, 1 ¼" may be assumed. The area of the haunch may be used in calculating section properties, but it is common practice to conservatively ignore for purpose of section properties (haunch dead load must be taken into account). Appropriate consideration of the haunch is the responsibility of the load rating engineer.

45.6.3.2.1 Curvature and/or Kinked Girders

The effects of curvature shall be considered for all curved steel girder structures. For structures meeting the criteria specified in **LRFD [4.6.1.2.4]** or the **Curved Steel Girder Guide Specification [4.2]**, the structure may be analyzed as if it were straight. However, regardless of the degree of curvature, the effects of curvature on flange lateral bending must always be considered in the analysis, either directly through a refined analysis or through an approximate method as detailed in **LRFD [C4.6.1.2.4b]** or the **Curved Steel Girder Guide Specification [4.2.1]**. This is applicable to discretely braced flanges. If a flange is continuously braced (e.g. encased in concrete or anchored to deck by shear connectors) then it need not be considered. In determining the load rating, flange lateral bending stress shall be added to the major axis bending flange stress, utilizing the appropriate equations specified in LRFD. When using the Curved Steel Girder Guide Specification, flange lateral bending stress reduces the allowable flange stress.

45.6.3.2.2 Skew

Load rating of steel structures with discontinuous cross-frames, in conjunction with skews exceeding 20 degrees shall consider flange lateral bending stress, either directly through a refined analysis or using approximate values provided in **LRFD [C6.10.1]**. This requirement only applies to structures with multi-member cross frames (X or K-brace), and full depth diaphragms between girders. Flange lateral bending stress is most critical when the bottom flange is stiffened transversely (discretely braced). For structures with shorter single member diaphragms (e.g. C or MC-shapes) between girders, where the bottom flange is less restrained, the load rating need not consider flange lateral bending stress due to skew.

Flange lateral bending stress, whether due to skew or curvature, is handled the same in a load rating equation. Refer to the flange lateral bending discussion in 45.6.3.2.1 for more information.

45.6.3.2.3 Variable Girder Spacing (Flare)

Girder spacings may vary over the length of a given girder (flared girder configuration). Some analysis software may allow for a varying distribution factor along the length of the girder. This is the most accurate and thus preferred method for dealing with a flared girder layout.

Alternatively, conservative assumptions may be made regarding the live load distribution and the assigned dead load to the girder being analyzed. The rating engineer is responsible for determining the appropriate assumptions and for ensuring that they produce conservative results. The methods described in **LRFD [C4.6.2.2.1]** are acceptable. All assumptions made should be clearly noted in the calculations and in the load rating summary sheet (See 45.9.1).

If the girders are flared such that the ratio of change in girder spacing to span length is greater than or equal to 0.015, then a refined analysis may be required. Consult the Bureau of Structures Rating Unit for structures that meet this criteria.

45.6.3.3 Truss

45.6.3.3.1 Gusset Plates

WisDOT requires gusset plates to be load rated anytime the loads applied to a structure are altered (see 45.3). Gusset plates should also be evaluated with reports of any significant deterioration. Rating procedures shall follow those specified in the AASHTO MBE.

45.6.3.4 Bascule-Type Movable Bridges

Apply twice the normal dynamic impact factor to live loading of the end floorbeam per **AASHTO LRFD Movable Spec [2.4.1.2.4]**. The end floorbeam will likely control the load rating of bascule bridges built before 1980.

45.6.4 Timber

As a material, timber is unique in that material strengths are tied to the load rating methodology used for analysis (typically ASD or LRFR for timber). Because of this and because of the fact that design/rating specifications have changed through the years, the load rating engineer must carefully consider the appropriate material strengths to use for a given member. When referencing historic plans, WisDOT recommends using the plans to determine the type of material (species and grade), but then using contemporary sources (including tables in 0) to determine material strengths and for rating methodology.

Based on experience, WisDOT recommends evaluating timber superstructures for posting vehicles when the rating factor falls below 1.25 instead of the typical 1.0. See 45.10 for more information on load posting.

45.6.4.1 Timber Slab

For longitudinal nail laminated slab bridges, the wheel load shall be distributed to a strip width equal to:

(wheel width) + 2x(deck thickness).

On bridges that are showing lamination slippage, breakage on the underside, or loose stiffener beam connections, the strip width shall be reduced to:

(wheel width) + 1x(deck thickness).

45.7 WisDOT Load Rating Policy and Procedure – Substructure

45.7.1 Timber Pile Abutments and Bents

Any decay or damage will result in the reduction of the load-carrying capacity based on a loss of cross-sectional area (for shear and compression) or in a reduction of the section modulus (for moment). The capacity of damaged timber bents will be based on the remaining cross-sectional area of the pile and the column stability factor (C_p) using “d”, the least remaining dimension of the column. Such reductions will be determined by the rating engineer based on field measurements, when available.

Timber piles with significant deterioration and/or tipping shall be load rated with consideration of lateral earth pressure and redundancy. Piles shall be assumed to be fixed 6 feet below the stream bed or ground line and pinned at their tops.