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2.1 Organizational Charts

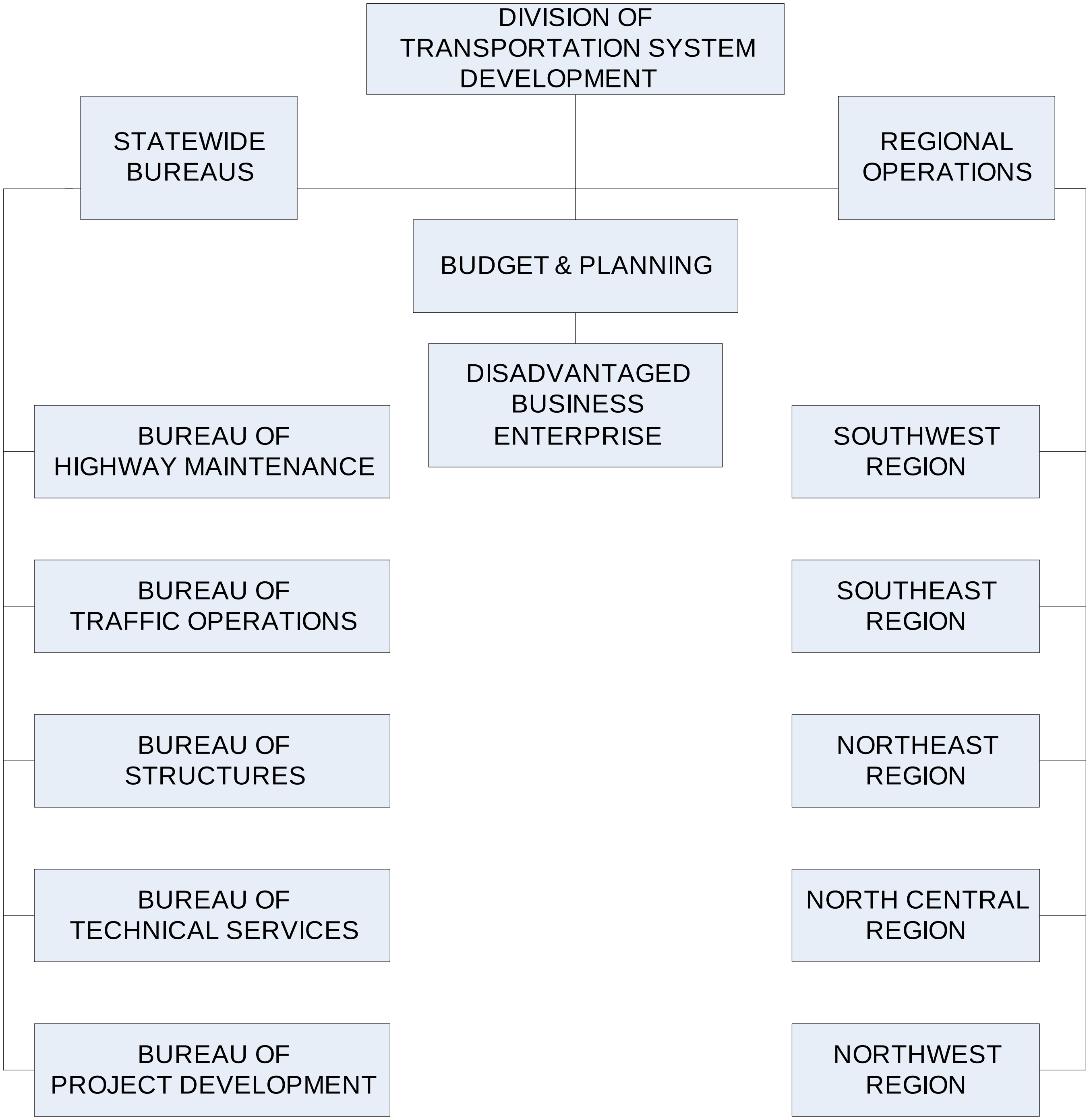

Figure 2.1-1
Division of Transportation System Development

2.4 Bridge Standards and Insert Sheets

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

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

1. FHWA Approval of Structure Standards Process

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

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

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

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

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

2.5 Bridge Numbers

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

WisDOT Policy Item:

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

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

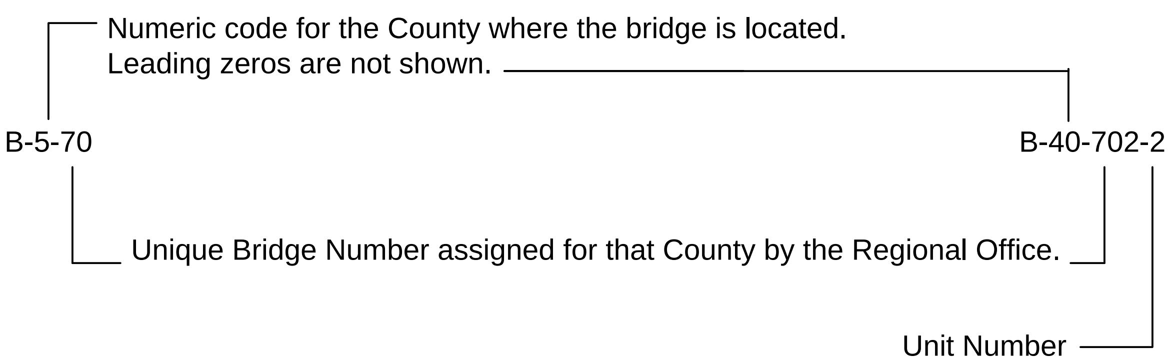

Figure 2.5-1
Bridge Number Detail

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

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

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3.1 Specifications and Standards

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

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

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

3.2 Geometrics and Loading

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

For highway structures, the minimum desirable longitudinal vertical gradient is 0.5 percent. There have been ponding problems on bridges with smaller gradients. This requirement is applied to the bridge in its final condition, without consideration of short term camber effects.

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

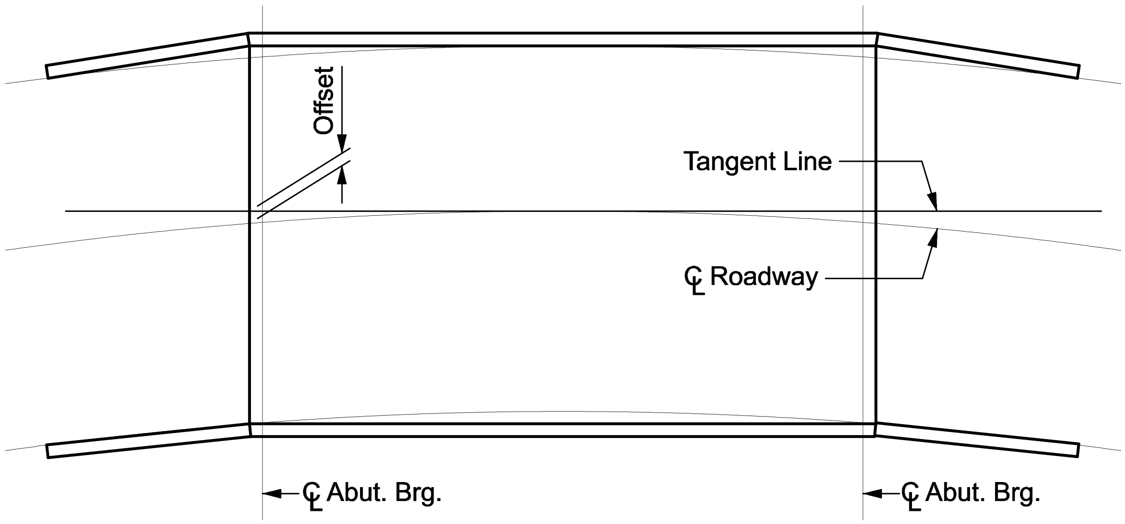
Highway bridge design live loadings follow the AASHTO LRFD Design specifications using HL93. Chapter 17 provides more detail on applying this load for design. WisDOT requires a specific vehicle design check using the Wis-SPV (Standard Permit Vehicle) which can be found in Chapter 45.

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

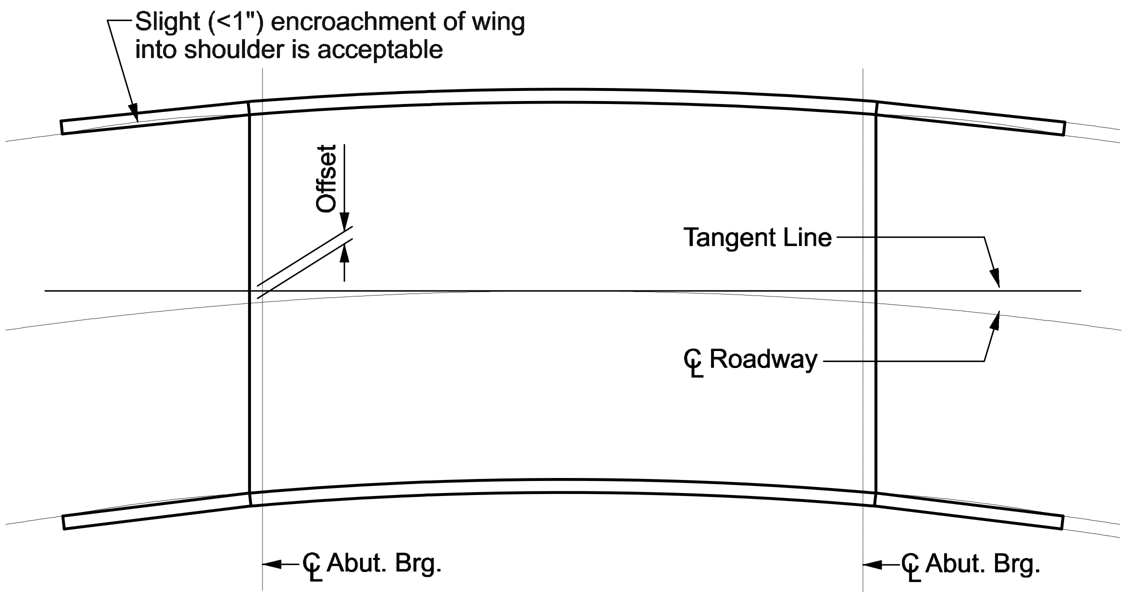
All new bridges constructed in the State of Wisconsin are designed for the clearances shown in FDM Procedure 11-35-1, Attachment 8. FDM Procedure 11-35-1, Attachment 9 covers the cases described in that section as well as bridge widenings. Wires and cables over highways are designed for clearances of 18’-0” to 22’-0”. Vertical clearance is needed for the entire roadway width (critical point to include traveled way, auxiliary lanes, turn lanes and shoulders).

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

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



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



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

Figure 3.2-1
Bridge Layout on Horizontal Curves

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5.1 Factors Governing Bridge Costs

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

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

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

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

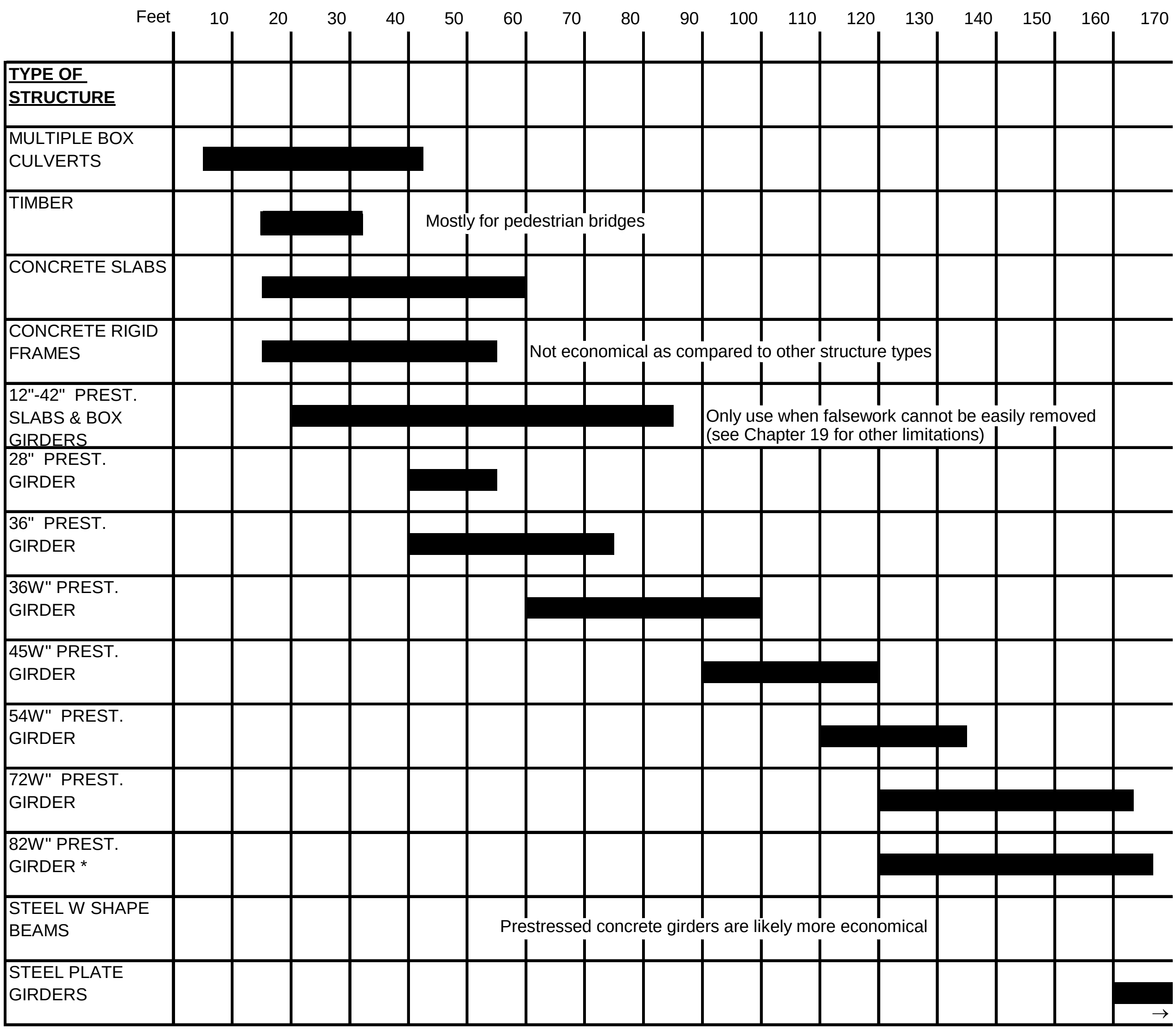
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

No costs are shown for rolled steel sections as these structures are not built very often. They have been replaced with prestressed girders which are usually more economical. The cost of plate girders is used to estimate rolled section costs.

For structures over a railroad, use the costs of grade separation structures. Costs vary considerably for railroad structures over a highway due to different railroad specifications.

Note: Current costs are given in English units.

5.2 Economic Span Lengths



*Currently there is a moratorium on the use of 82W" prestressed girders in Wisconsin

Figure 5.2-1
Economic Span Lengths

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
206.6010.S	Temporary Shoring	LS	--
210.0100	Backfill Structure	CY	22.49
303.0115	Pit Run	CY	9.63
311.0115	Breaker Run	CY	23.50
502.0100	Concrete Masonry Bridges	CY	460.00
502.1100	Concrete Masonry Seal	CY	153.00
502.2000	Compression Joint Sealer Preformed Elastomeric (width)	LF	42.12
502.3100	Expansion Device (structure) (LS)	LF	158.73
502.3110.S	Expansion Device Modular (structure) (LS)	LF	
502.3200	Protective Surface Treatment	SY	2.35
502.6500	Protective Coating Clear	GAL	65.00
503.0128	Prestressed Girder Type I 28-Inch	LF	97.82
503.0136	Prestressed Girder Type I 36-Inch	LF	148.35
503.0137	Prestressed Girders Type I 36W-Inch	LF	156.50
503.0145	Prestressed Girder Type I 45-Inch	LF	162.29
503.0146	Prestressed Girders Type I 45W-Inch	LF	180.85
503.0154	Prestressed Girder Type I 54-Inch	LF	
503.0155	Prestressed Girder Type I 54W-Inch	LF	178.03
503.0170	Prestressed Girder Type I 70-Inch	LF	
503.0172	Prestressed Girders Type I 72W-Inch	LF	183.14
503.0182	Prestressed Girder Type I 82W-Inch	LF	
504.0100	Concrete Masonry Culverts	CY	439.50
504.0500	Concrete Masonry Retaining Walls	CY	461.74
505.0405	Bar Steel Reinforcement HS Bridges	LB	0.86
506.2605	Bar Steel Reinforcement HS Culverts	LB	0.56
506.2610	Bar Steel Reinforcement HS Retaining Walls	LB	0.92
506.3005	Bar Steel Reinforcement HS Coated Bridges	LB	0.91
506.3010	Bar Steel Reinforcement HS Coated Culverts	LB	1.38
506.3015	Bar Steel Reinforcement HS Coated Retaining Walls	LB	0.96
506.0105	Structural Carbon Steel	LB	4.57
506.0605	Structural Steel HS	LB	1.48
506.2605	Bearing Pads Elastomeric Non-Laminated	EACH	63.22
506.2610	Bearing Pads Elastomeric Laminated	EACH	729.90
506.3005	Welded Shear Stud Connectors 7/8 x 4-Inch	EACH	3.50
506.3010	Welded Shear Stud Connectors 7/8 x 5-Inch	EACH	4.83
506.3015	Welded Shear Stud Connectors 7/8 x 6-Inch	EACH	3.54
506.3020	Welded Shear Stud Connectors 7/8 x 7-Inch	EACH	5.18
506.3025	Welded Shear Stud Connectors 7/8 x 8-Inch	EACH	3.82
506.4000	Steel Diaphragms (structure)	EACH	500.20
506.5000	Bearing Assemblies Fixed (structure)	EACH	958.92
506.6000	Bearing Assemblies Expansion (structure)	EACH	1,526.50
507.0200	Treated Lumber and Timber	MBM	
508.1600	Piling Treated Timber Delivered	LF	
510.2005	Preboring Cast-in-Place Concrete Piling	LF	27.36
510.3021	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.219-Inch	LF	32.44

510.3030	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.25-Inch	LF	33.20
510.3040	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.365-Inch	LF	36.52
510.3050	Piling CIP Concrete Delivered and Driven 10 ¾ x 0.5-Inch	LF	--
510.3023	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.219-Inch	LF	
510.3033	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.25-Inch	LF	34.38
510.3043	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.375-Inch	LF	39.01
510.3053	Piling CIP Concrete Delivered and Driven 12 ¾ x 0.5-Inch	LF	
510.3024	Piling CIP Concrete Delivered and Driven 14 x 0.219-Inch	LF	--
510.3034	Piling CIP Concrete Delivered and Driven 14 x 0.25-Inch	LF	
510.3044	Piling CIP Concrete Delivered and Driven 14 x 0.375-Inch	LF	
510.3054	Piling CIP Concrete Delivered and Driven 14 x 0.5-Inch	LF	
510.3026	Piling CIP Concrete Delivered and Driven 16 x 0.219-Inch	LF	--
510.3036	Piling CIP Concrete Delivered and Driven 16 x 0.25-Inch	LF	--
510.3046	Piling CIP Concrete Delivered and Driven 16 x 0.375-Inch	LF	--
510.3056	Piling CIP Concrete Delivered and Driven 16 x 0.5-Inch	LF	
511.2105	Piling Steel Delivered and Driven HP 10-Inch x 42 LB	LF	31.48
511.2110	Piling Steel Delivered and Driven HP 12-Inch x 53 LB	LF	40.18
511.2115	Piling Steel Delivered and Driven HP 12-Inch x 74 LB	LF	45.54
511.2120	Piling Steel Delivered and Driven HP 14-Inch x 73 LB	LF	
511.3000	Pile Points	EACH	80.11
511.6000	Piling Steel Preboring	LF	156.88
512.1000	Piling Steel Sheet Temporary	SF	15.00
513.4050	Railing Tubular Type F (structure) (LS)	LF	108.63
513.4052 or 3	Railing Tubular Type F- (4 or 5) Modified (structure) (LS)	LF	140.94
513.4055	Railing Tubular Type H (structure) (LS)	LF	110.96
513.4060	Railing Tubular Type M (structure) (LS)	LF	169.63
513.4065	Railing Tubular Type PF (structure) (LS)	LF	--
513.4090	Railing Tubular Screening Structure B-	LF	104.77
513.7050	Railing Type W Structure B-	LF	123.26
00000	Concrete Railing, "Texas Rail"	LF	160.00
00000	Concrete Parapet, Type 'LF' & 'A' (estimate)	LF	80.00
513.7005	Railing Steel Type C1 (structure) (LS)	LF	71.39
513.7010	Railing Steel Type C2 (structure) (LS)	LF	180.25
513.7015	Railing Steel Type C3 (structure) (LS)	LF	118.48
513.7020	Railing Steel Type C4 (structure) (LS)	LF	38.19
513.7025	Railing Steel Type C5 (structure) (LS)	LF	
513.7030	Railing Steel Type C6 (structure) (LS)	LF	114.03
514.0445	Floor Drains Type GC	EACH	1,598.00
514.2625	Downspouts 6-Inch	LF	--

516.0500	Rubberized Membrane Waterproofing	SY	28.74
517.1010.S	Concrete Staining	SF	1.25
517.1015.S	Concrete Staining Multi-Color	SF	4.67
517.1050.S	Architectural Surface Treatment	SF	4.78
604.0400	Slope Paving Concrete	SY	46.66
604.0500	Slope Paving Crushed Aggregate	SY	17.21
604.0600	Slope Paving Select Crushed Material	SY	16.88
606.0100	Riprap Light	CY	
606.0200	Riprap Medium	CY	57.00
606.0300	Riprap Heavy	CY	43.50
606.0500	Grouted Riprap Light	CY	100.00
606.0600	Grouted Riprap Medium	CY	
606.0700	Grouted Riprap Heavy	CY	78.61
612.0106	Pipe Underdrain 6-Inch	LF	6.66
612.0206	Pipe Underdrain Unperforated 6-Inch	LF	7.41
612.0406	Pipe Underdrain Wrapped 6-Inch	LF	7.80
616.0205	Fence Chain Link 5-FT	LF	43.50
616.0206	Fence Chain Link 6-FT	LF	
616.0208	Fence Chain Link 8-FT	LF	--
645.0105	Geotextile Fabric Type C	SY	2.78
645.0111	Geotextile Fabric Type DF Schedule A	SY	1.42
645.0120	Geotextile Fabric Type HR	SY	3.03
652.0125	Conduit Rigid Metallic 2-Inch	LF	19.00
652.0135	Conduit Rigid Metallic 3-Inch	LF	21.75
652.0225	Conduit Rigid Nonmetallic Schedule 40 2-Inch	LF	4.57
652.0235	Conduit Rigid Nonmetallic Schedule 40 3-Inch	LF	4.86
SPV.0035	HPC Masonry Superstructure	CY	505.00
SPV.0085	Stainless Steel Reinforcement	LB	4.04
SPV.0165	Anti-Graffiti Coating	SF	2.23
SPV.0180	Pigmented Protective Surface Treatment	SY	12.40
SPV.0180	Protective Polymer Coating	SY	43.50

Table 5.3-1

Contract Unit Bid Prices for New Structures

Item No.	Bid Item	Unit	Cost
455.0___	Asphaltic Material ________	TON	97.00
460.1___	HMA Pavement Type _____	TON	23.60
502.5002	Masonry Anchors Type L No. 4 Bars	EACH	7.47
502.5005	Masonry Anchors Type L No. 5 Bars	EACH	10.71
502.5010	Masonry Anchors Type L No. 6 Bars	EACH	12.654
502.5015	Masonry Anchors Type L No. 7 Bars	EACH	12.00
502.5020	Masonry Anchors Type L No. 8 Bars	EACH	71.34
502.5025	Masonry Anchors Type L No. 9 Bars	EACH	
502.6102	Masonry Anchors Type S ½-Inch	EACH	13.85
502.6105	Masonry Anchors Type S 5/8-Inch	EACH	18.39
502.6110	Masonry Anchors Type S ¾-Inch	EACH	15.10
502.6115	Masonry Anchors Type S 7/8-Inch	EACH	--
502.6120	Masonry Anchors Type S 1-Inch	EACH	
505.0904	Bar Couplers No. 4	EACH	13.33
505.0905	Bar Couplers No. 5	EACH	21.94
505.0906	Bar Couplers No. 6	EACH	25.69
505.0907	Bar Couplers No. 7	EACH	21.76
505.0908	Bar Couplers No. 8	EACH	49.24
505.0909	Bar Couplers No. 9	EACH	59.09
509.0301	Preparation Decks Type 1	SY	66.00
509.0302	Preparation Decks Type 2	SY	68.50
509.0500	Cleaning Decks	SY	16.90
509.1000	Joint Repair	SY	641.50
509.1200	Curb Repair	LF	40.00
509.1500	Concrete Surface Repair	SF	81.12
509.2000	Full-Depth Deck Repair	SY	212.50
509.2500	Concrete Masonry Overlay Decks	CY	470.00
SPV.0165	Epoxy Overlay	SF	4.76

Table 5.3-2
Contract Unit Bid Prices for Rehab Structures

5.4 Bid Letting Cost Data

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

5.4.1 2007 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	48	529,222	44,292,799	43.38	83.69
Reinf. Conc. Slabs (All But A5)	50	133,565	14,226,156	47.96	98.98
Reinf. Conc. Slabs (A5 Abuts)	24	40,309	4,286,494	47.85	92.14
Prestressed Box Girders	3	11,522	1,350,270	68.84	117.19

Table 5.4-6
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	24	218,501	16,506,445	55.79	75.54
Steel Plate Girders	1	32,000	4,959,856	122.43	155.00
Reinf. Conc. Slabs (All But A5)	2	9,738	709,168	32.77	72.82
Reinf. Conc. Slabs (A5 Abuts)	1	1,944	226,433	47.81	116.46

Table 5.4-7
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	5	1,168.00
Twin Cell	9	1,000.00
Triple Cell	1	3,832.00
Precast Box	1	894.00

Table 5.4-8
Box Culverts

Pedestrian Bridge	Cost per Sq. Ft.
B-13-605	154.34
B-45-96	443.41

Table 5.4-9
Pedestrian Bridges

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Walls	6	14,133	752,236	53.23
Modular Walls	0	0	0	0
Concrete Walls	6	21,376	1,254,180	58.67
Panel Walls	0	0	0	0

Table 5.4-10
Retaining Walls

5.4.2 2008 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	27	354,319	37,636,697	56.00	106.00
Reinf. Conc. Slabs (All But A5)	28	29,381	3,892,609	45.00	98.50
Reinf. Conc. Slabs (A5 Abuts)	20	19,900	2,529,658	53.50	127.00
Prestressed Box Girders	1	762	106,847	109.00	140.00

Table 5.4-11
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	68	617,067	52,412,539	64.50	85.00
Steel Plate Girders	0	--	--	--	--
Reinf. Conc. Slabs (All But A5)	2	23,777	2,769,953	58.50	116.50
Reinf. Conc. Slabs (A5 Abuts)	0	--	--	--	--

Table 5.4-12
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	7	1,059.00
Twin Cell	15	1,914.00
Triple Cell	0	--
Aluminum	0	--

Table 5.4-13
Box Culverts

Pedestrian Bridges	Cost per Sq. Ft.
None this Year	--

Table 5.4-14
Pedestrian Bridges

Railroad Bridges	Cost per Sq. Ft.
None this Year	--

Table 5.4-15
Railroad Bridges

Bascule Bridge	Cost per Sq. Ft.
None this Year	--

Table 5.4-16
Bascule Bridges

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Walls	4	14,292	520,912	36.50
Modular Walls	0	--	--	--
Concrete Walls	14	23,572	2,572,658	108.00
Panel Walls	5	11,939	782,972	65.50

Table 5.4-17
Retaining Walls

5.4.3 2009 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	27	225,572	23,546,996	54.77	104.39
Reinf. Conc. Slabs (All but A5)	39	108,422	11,214,819	46.46	103.44
Reinf. Conc. Slabs (A5 Abuts)	32	58,049	6,312,845	51.00	108.75
Prestressed Box Girders	0				

Table 5.4-18
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	124	776,329	67,163,261	50.71	86.51
Steel Plate Girders	2	21,725	4,038,011	114.36	185.87
Reinf. Conc. Slabs (All but A5)	0				
Reinf. Conc. Slabs (A5 Abuts)	0				
Steel I-Beam	4	34,227	3,454,905	58.22	100.94
Arch Structures	2	4,750	1,637,760	0	344.79
Pedestrian Stuctures	1	2,286	1,712,743	0	749.23

Table 5.4-19
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	16	1,470.36
Twin Cell	11	2,331.10
Triple Cell	1	6,922.41
Pipe	2	1,072.73

Table 5.4-20
Box Culverts

Pedestrian Bridges	Cost per Sq. Ft.
None this Year	--

Table 5.4-21
Pedestrian Bridges

Timber Bridges	Cost per Sq. Ft.
B-9-285 (County Built)	47.20

Table 5.4-22
County Timber Bridges

Bascule Bridge	Cost per Sq. Ft.
None this Year	--

Table 5.4-23
Bascule Bridges

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Walls	26	103,486	5,460,180	52.76
Modular Walls	0			
Concrete Walls	6	25,025	1,109,328	44.33
Panel Walls	2	5,873	863,092	146.96
Pile Walls	5	168,403	2,930,175	17.40

Table 5.4-24
Retaining Walls

5.4.4 2010 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	20	255,157	23,302,014	58.02	91.32
Reinf. Conc. Slabs (All but A5)	24	60,992	6,851,861	61.34	112.34
Reinf. Conc. Slabs (A5 Abuts)	25	54,354	6,988,519	70.10	128.57
Prestressed Box Girders	1	3,351	463,639	78.97	138.36

Table 5.4-25
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	31	315,515	25,858,760	58.18	81.96
Steel Plate Girders	4	71,510	21,217,890	99.42	296.71
Reinf. Conc. Slabs (All but A5)	20	168,719	13,881,152	36.77	82.27
Reinf. Conc. Slabs (A5 Abuts)	0				
Trapezoid Box	3	82,733	10,546,181	89.12	127.50

Table 5.4-26
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	22	1,718.00
Twin Cell	8	1,906.00
Triple Cell	1	928.00
Pipe	1	1,095.00

Table 5.4-27
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
B-23-61	133.90

Table 5.4-28
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
4	179.56

Table 5.4-29
Pedestrian Bridges

Bascule Bridge	Cost per Sq. Ft.
None this Year	--

Table 5.4-30
Bascule Bridges

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Walls	74	448,972	26,243,005	58.45
Modular Walls	0			
Concrete Walls	6	38,680	2,223,277	57.48
Panel Walls	17	113,113	11,827,963	104.57
Tangent Pile Walls	4	36,974	2,347,442	63.49
Wired Faced MSE Wall	2	22,130	907,330	41.00
Secant Wall	1	8,500	913,292	107.45
Soldier Pile Wall	3	251,344	4,448,344	17.72

Table 5.4-31
Retaining Walls

5.4.5 2011 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	36	218,311	18,719,353	50.45	85.75
Reinf. Conc. Slabs (All but A5)	22	63,846	7,135,430	52.90	111.76
Reinf. Conc. Slabs (A5 Abuts)	14	21,005	2,470,129	53.00	117.60
Prestressed Box Girders	0				

Table 5.4-25
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	44	337,346	31,596,585	65.90	93.66
Steel Plate Girders	0				
Reinf. Conc. Slabs (All but A5)	6	33,787	3,462,995	52.90	102.49
Reinf. Conc. Slabs (A5 Abuts)	0				
Trapezoid Box	0				

Table 5.4-26
Grade Separation Structures

Box Culverts	No. of Culverts	Cost per Lin. Ft.
Single Cell	5	2,140.00
Twin Cell	6	1,998.00
Triple Cell	5	3,518.00
Precast	1	7,385.00

Table 5.4-27
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
None this Year	--

Table 5.4-28
Pre-Fab Pedestrian Bridge

Pedestrian Bridges	Cost per Sq. Ft.
None this Year	--

Table 5.4-29
Pedestrian Bridges

Railroad Bridge	Cost per Sq. Ft.
B-20-210	3,654.30

Table 5.4-30
Bascule Bridges

Retaining Walls	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	6	7,893	494,274	62.62
MSE Panel Walls	19	87,000	6,679,782	76.78
Modular Walls	0			
Concrete Walls	3	3,516	237,230	67.47
Panel Walls	2	14,832	3,458,722	233.19
Tangent Pile Walls	3	10,139	1,581,071	155.94
Wired Faced MSE Wall	18	149,735	11,412,474	76.22
Secant Wall	0			
Soldier Pile Wall	2	7,849	779,563	99.32

Table 5.4-31
Retaining Walls

Table of Contents

14.1 Introduction

Retaining walls are used to provide lateral resistance for a mass of earth or other material to accommodate a transportation facility. These walls are used in a variety of applications including right-of-way restrictions, protection of existing structures that must remain in place, grade separations, new highway embankment construction, roadway widening, stabilization of slopes, protection of environmentally sensitive areas, staging, and temporary support including excavation or underwater construction support, etc.

Several types of retaining wall systems are available to retain earth and meet specific project requirements. Many of these wall systems are proprietary wall systems while others non-proprietary or design-build in Wisconsin. The wall selection criteria and design policies presented in this chapter are to ensure consistency of standards and applications used throughout WisDOT projects.

14.1.1 Wall Development Process

Overall, the wall development process requires an iterative collaboration between WisDOT Regions, Structures Design Section, Geotechnical Unit and WisDOT Consultants.

Retaining wall development is described in Section 11-55-5 of the *Facilities Development Manual*. WisDOT Regional staff determines the need for permanent retaining walls on highway projects. A wall number is assigned as per criteria discussed in 14.1.1.1 of this chapter. The Regional staff prepares a Structures Survey Report (SSR) that includes a preliminary evaluation of wall type, location, and height including a preliminary layout plan.

Based on the SSR, a Geotechnical site investigation may be required to determine foundation and retained soil properties. A hydraulic analysis is also conducted, if required, to asses scour potential. The Geotechnical investigation generally includes a subsurface and laboratory investigation. For the departmental-designed walls, the Bureau of Technical Services, Foundation & Pavement Unit (Geotechnical Unit) can recommend the scope of soil exploration needed and provide/recommend bearing resistance, overall stability, and settlement of walls based on the geotechnical exploration results.

The SSR is sent to the wall designer (Structures Design Section or WisDOT’s Consultant) for wall selection, design and contract plan preparation. Based on the wall selection criteria discussed in 14.3, either a proprietary or a non-proprietary wall system is selected.

Proprietary walls, as defined in 14.2, are pre-approved by the WisDOT’s Structures Design Section. Preapproval process for the proprietary walls is explained in 14.16. The structural design, internal and final external stability of proprietary wall systems are the responsibility of the supplier/contractor. The design and shop drawing computations of the proprietary wall systems are also reviewed by the Structures Design Section in accordance with the plans and special provisions. The preliminary external stability, overall stability and settlement computations of these walls are performed by the Geotechnical Unit or the WisDOT’s Consultant. Design and shop drawings must be approved by the Structures Design Section prior to start of the construction. Design of all temporary walls is the responsibility of the contractor.

Non-proprietary retaining walls are designed by WisDOT or its Consultant. The internal stability and the structural design of such walls are performed by the Structures Design Section or WisDOT's Consultant. The external and overall stability is performed by the Geotechnical Engineering Unit or Geotechnical Engineer of record.

The final contract plans of retaining walls include final plans, details, special provisions, contract requirements, and cost estimate for construction. The Subsurface Exploration is part of the final contract plans.

The wall types and wall selection criteria to be used in wall selection are discussed in 14.2 and 14.3 of this chapter respectively. General design concepts of a retaining wall system are discussed in 14.4. Design criteria for specific wall systems are discussed in sections 14.5 thru 14.11. The plan preparation process is briefly described in Chapter 2 – General and Chapter 6 – Plan Preparation. The contract documents and contract requirements are discussed in 14.14 and 14.15 respectively.

For further information related to wall selection, design, approval process, pre-approval and review of proprietary wall systems please contact Structures Design Section of the Bureau of Structures at 608-266-8489. For questions pertaining to geotechnical analyses and geotechnical investigations please contact the Geotechnical Unit at 608-246-7940.

14.1.1.1 Wall Numbering System

Permanent retaining walls that are designed for a design life of 75 years or more should be identified by a wall number, R-XX-XXX, as assigned by the Region. For a continuous wall consisting of various wall types, the numbering system should include unit numbers so that the numbering appears as R-XX-XXX-001, R-XX-XXX-002, and so on. The first two digits represent the county the wall is located in and the next set(s) of digits represent the undivided wall.

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

- Modular block gravity walls and MSE walls: If the top of leveling pad to top of wall height (including any coping) exceeds 5.0 ft. at any point along the wall length.
- Cast-in-place, sheet pile, and all other walls: If the exposed height from the plan ground line to top of wall (including any coping) exceeds 5.0 ft. at any point along the wall length.

Cast-in-place walls being utilized strictly as bridge abutment wings do not require R numbers as they are considered part of the bridge.

CIP cantilever wall with sloping surcharge

For CIP cantilever walls, lateral active earth pressure shall be computed using Coulomb's theory for short heels or using Rankine theory for very long heels in accordance with the criteria presented in **LRFD [3.11.5.3]** and **LRFD [C3.11.5.3]**.

Walls resting on rock or batter piles can be designed for active earth pressure, based on WisDOT policy and in accordance with **LRFD [3.11.5.2].** Effect of the passive earth pressure on the front face of the wall shall be neglected in stability computation, unless the base of the wall extends below depth of maximum scour, freeze thaw or other disturbances in accordance with **LRFD [11.6.3.5]**.

Effect of surcharge loads ES present at the surface of the backfill of the wall shall be included in the analysis in accordance with 14.4.5.4.1. Walls with horizontal backfill shall be designed for live load surcharge in accordance with 14.4.5.4.2.

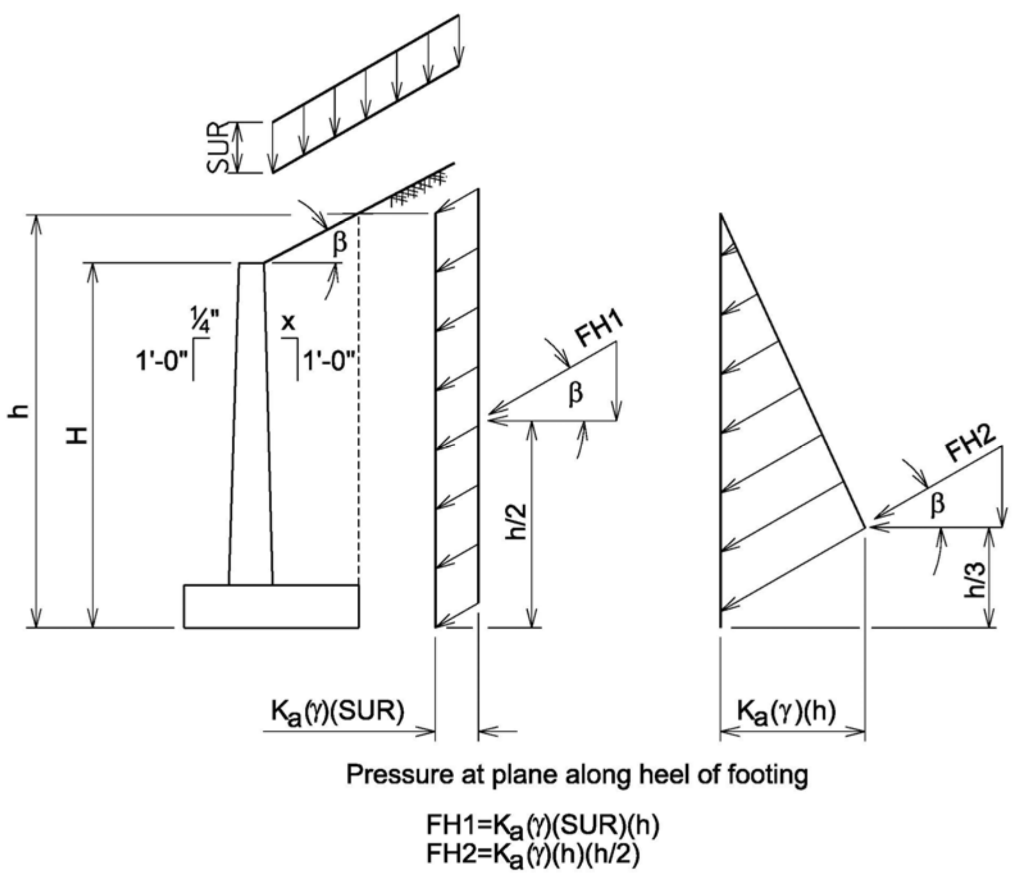


Figure14.4-1
Loading Diagram for a Cantilever Retaining Wall with Surcharge Loading

MSE Walls

The loading and earth pressure diagram for an MSE wall shall be developed in accordance with **LRFD [11.11.2]** and described below for the three conditions defined earlier in this section.

MSE Wall with Horizontal Backslope and Traffic Surcharge

Figure 14.4-2 shows a procedure to estimate the earth pressure. The active earth pressure for horizontal backslope is computed using a simplified version of Coulomb theory

$$K_a = \tan^2(45 - \phi_f / 2)$$

Where:

K_a = Coefficient of active earth pressure

φ_f = Angle of internal friction of retained earth

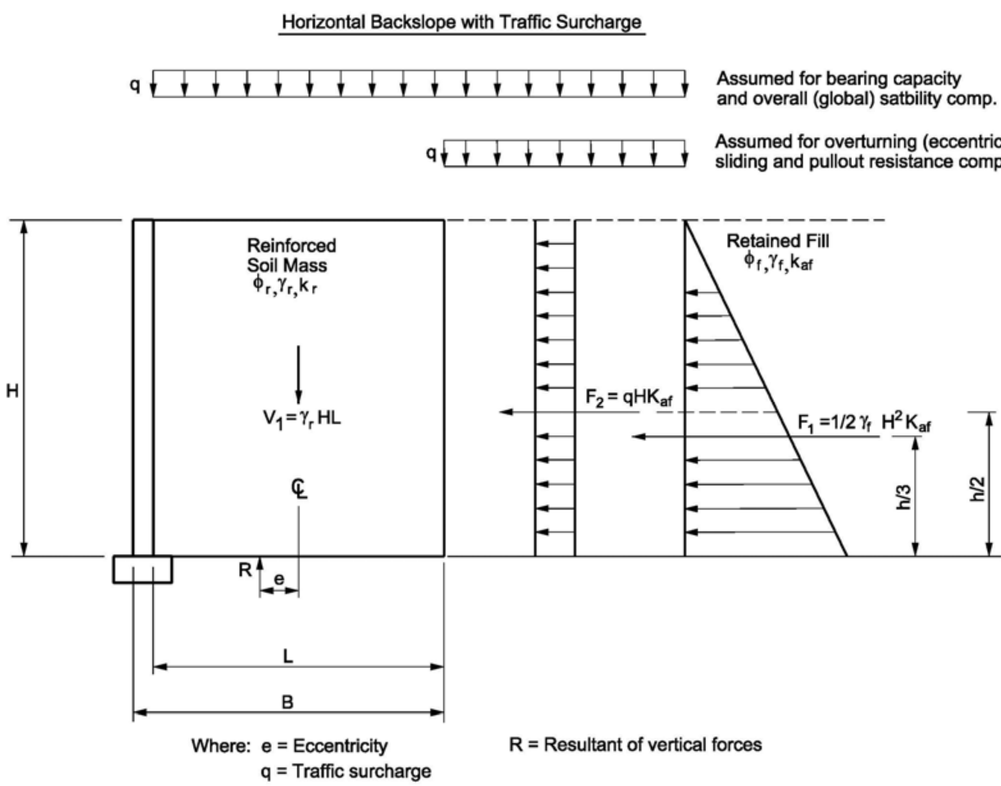


Figure 14.4-2
MSE Walls Earth Pressure for Horizontal Backslope with Traffic Surcharge
(Source *AASHTO LRFD*)

MSE Wall with Sloping Surcharge

The active earth pressure coefficient K_a is computed using Coulomb's equation. The force on the rear of the reinforced soil mass (F_t) and the resulting horizontal (F_h) and vertical (F_v) forces are determined from the following equations:

$$F_T = 1/2\ \gamma_f h^2 K_{af}$$

$$F_h = F_t \cos \beta$$

$$F_v = F_t \sin \beta$$

Where:

γ_f	=	Unit weight of retained fill material
β	=	Slope angle of backfill behind wall
δ	=	Angle of friction between retained backfill and reinforced backfill
h	=	See Figure 14.4-3
K_{af}	=	Use Coulomb's equation

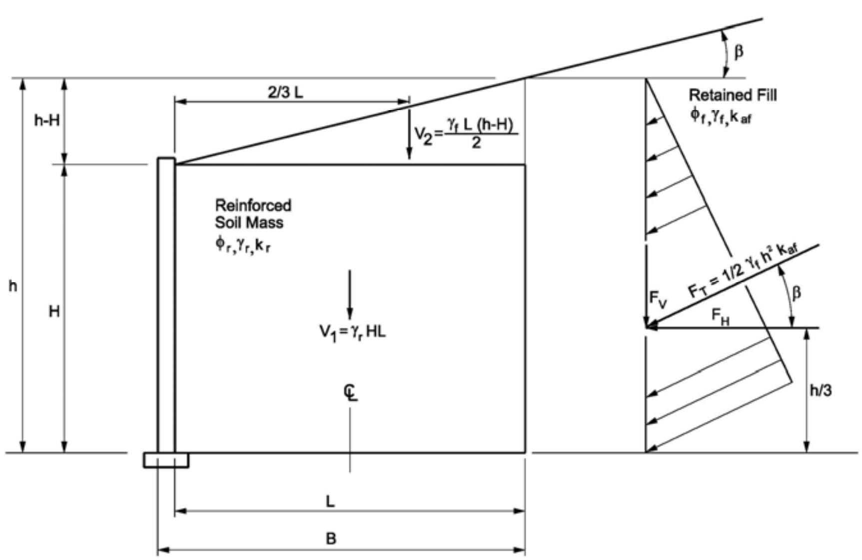


Figure 14.4-3
MSE Walls Earth Pressure for Sloping Backfill
(Source *AASHTO LRFD*)

MSE Wall with Broken Backslope

For broken backslopes, the active earth pressure coefficient is determined using Coulomb's equation except that surcharge angle β and interface angle δ is substituted with infinite slope angle I. Force, F_t, is determined using:

$F_t = 1/2\ \gamma_f h^2 K_{af}$

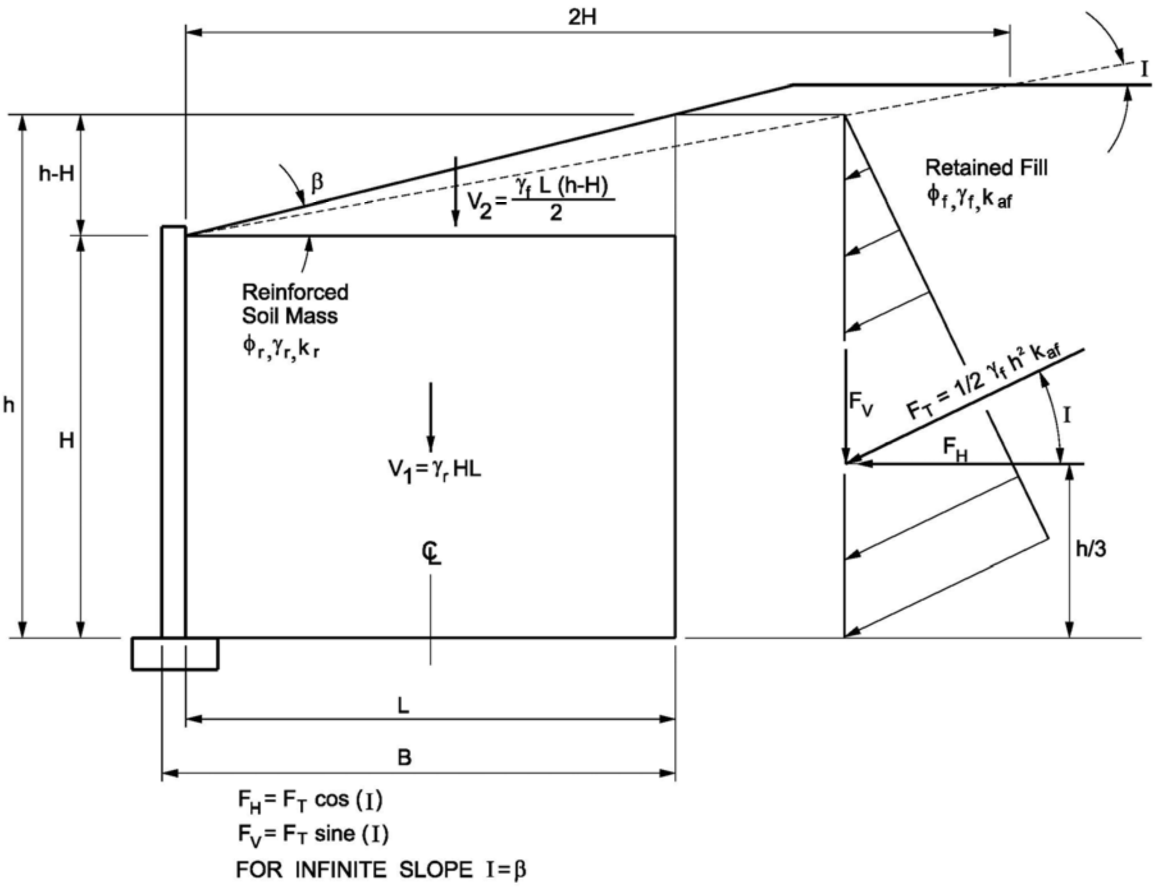


Figure 14.4-4
MSE Walls Earth Pressure for Broken Backfill
(Source *AASHTO LRFD*)

Modular Block Gravity Wall with Sloping Surcharge

When designing a "Modular Block Gravity Wall" without setback and with level backfill, the active earth pressure coefficient may be determined using Rankine theory from the following formula.

$K_a = \tan^2 (45 - \phi_f / 2)$

14.4.5.5 Load factors and Load Combinations

The nominal loads and moments as described in 14.4.5.4.5 are factored using load factors found in **LRFD [Tables 3.4.1-1 and 3.4.1-2]**. The load factors applicable for most wall types considered in this chapter are given in Table 14.4-1. Load factors are selected to produce a total extreme factored force effect, and for each loading combination, both maximum and minimum extremes are investigated as part of the stability check, depending upon the expected wall failure mechanism.

Direction of Load	Load Type	Load Factor, γ_i		
		Strength I Limit		Service I Limit
		Maximum	Minimum	
Load Factors for Vertical Loads	Dead Load of Structural Components and Non-structural attachments DC	1.25	0.90	1.00
	Earth Surcharge Load ES	1.50	0.75	1.00
	Vertical Earth Load EV	1.35	1.00	1.00
	Water Load WA	1.00	1.00	1.00
	Live Load Surcharge LS	1.75	0.0	1.00
	Dead Load of Wearing Surfaces and Utilities DW	1.50	0.65	1.00
Load Factors for Horizontal Loads	Horizontal Earth Pressure EH			
	Active	1.50	0.90	1.00
	At-Rest	1.35	0.90	1.00
	Passive	1.35	NA	1.00
	Earth Surcharge ES	1.50	0.75	1.00
	Live Load Surcharge LS	1.75	1.75	1.00

Table 14.4-1
Load Factors

The factored loads are grouped to consider the force effect of all loads and load combinations for the specified load limit state in accordance with **LRFD [3.4.1].** Figure 14.4-8 illustrates the load factors and load combinations applicable for checking sliding stability and eccentricity for a cantilever wall at the Strength I limit state. This figure shows that structure weight DC is factored by using a load factor of 0.9 and the vertical earth load EV is factored by using a factor of 1.0. This causes contributing stabilizing forces against sliding to have a minimum force effect. At the same time, the horizontal earth load is factored by 1.5 resulting in maximum force effect for computing sliding at the base.

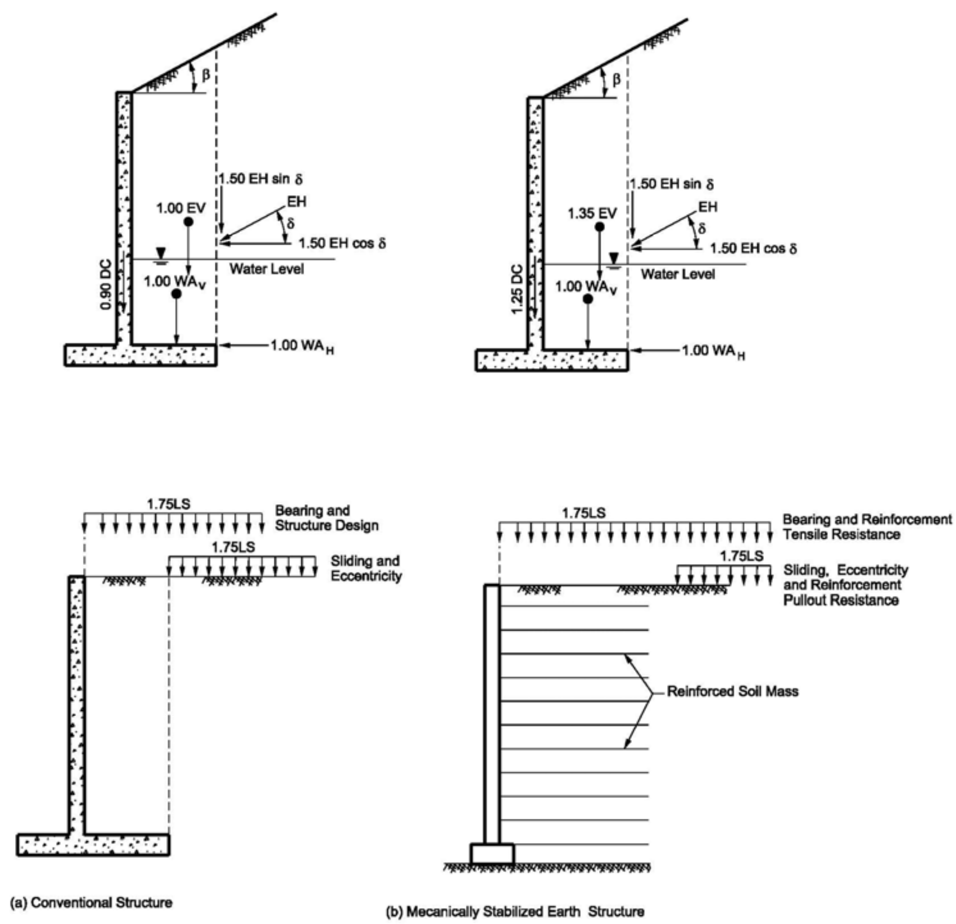


Figure 14.4-8
Application of Load Factors
(Source *AASHTO LRFD*)

14.4.5.6 Resistance Requirements and Resistance Factors

The wall components shall be proportioned by the appropriate methods so that the factored resistance as shown in **LRFD [1.3.2.1-1]** is no less than the factored loads, and satisfy criteria in accordance with **LRFD [11.5.4]** and **LRFD [11.6] thru [11.11]**. The factored resistance R_r is computed as follows: $R_r = \phi R_n$

Where

R_r = Factored resistance

R_n = Nominal resistance recommended in the Geotechnical Report

ϕ = Resistance factor

The resistance factors shall be selected in accordance with **LRFD [Tables 10.5.5.2.2-1, 10.5.5.2.3-1, 10.5.5.2.4-1, 11.5.6.1]**. Commonly used resistance factors for retaining walls are presented in Table 14.4-2.

14.4.6 Material Properties

The unit weight and strength properties of retained earth and foundation soil/rock (γ_f) are supplied in the geotechnical report and should be used for design purposes. Unless otherwise noted or recommended by the Designer or Geotechnical Engineer of record, the following material properties shall be assumed for the design and analysis if the selected backfill, concrete, and steel conforms to the WisDOT's *Standard Construction Specifications*:

Granular Backfill Soil Properties:

Internal Friction angle of backfill ϕ_f = 30 degrees

Backfill cohesion c = 0 psf

Unit Weight γ_f = 120 pcf

Concrete:

Compressive strength, f'_c at 28 days = 3500 psi

Unit Weight = 150 pcf

Steel reinforcement:

Yield strength f_y = 60,000 psi

Modulus of elasticity E_s = 29,000 ksi

Wall-Type and Condition		Resistance Factors
Mechanically Stabilized Earth Walls, Gravity Walls, and Semi-Gravity		
Bearing resistance	• Gravity & Semi-gravity • MSE	0.55 0.65
Sliding		1.00
Tensile resistance of metallic reinforcement and connectors	Strip reinforcement • Static loading Grid reinforcement • Static loading	0.75 0.65
Tensile resistance of geo-synthetic reinforcements and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
Prefabricated Modular Walls		
Bearing		**LRFD [10.5]**
Sliding		**LRFD [10.5]**
Passive resistance		**LRFD [10.5]**
Non-Gravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		**LRFD [10.5]**
Passive resistance of vertical elements		0.75
Pullout resistance of anchors	• Cohesionless soils • Cohesive soils • Rock	0.65 0.70 0.50
Pullout resistance of anchors	• Where proof tests are conducted	1.00
Tensile resistance of anchor tendons	• Mild steel • High strength steel	0.90 0.80
Flexural capacity of vertical elements		0.90

<u>Table 14.4-2</u>
Resistance Factors
Source **LRFD [Table 11.5.6-1]**

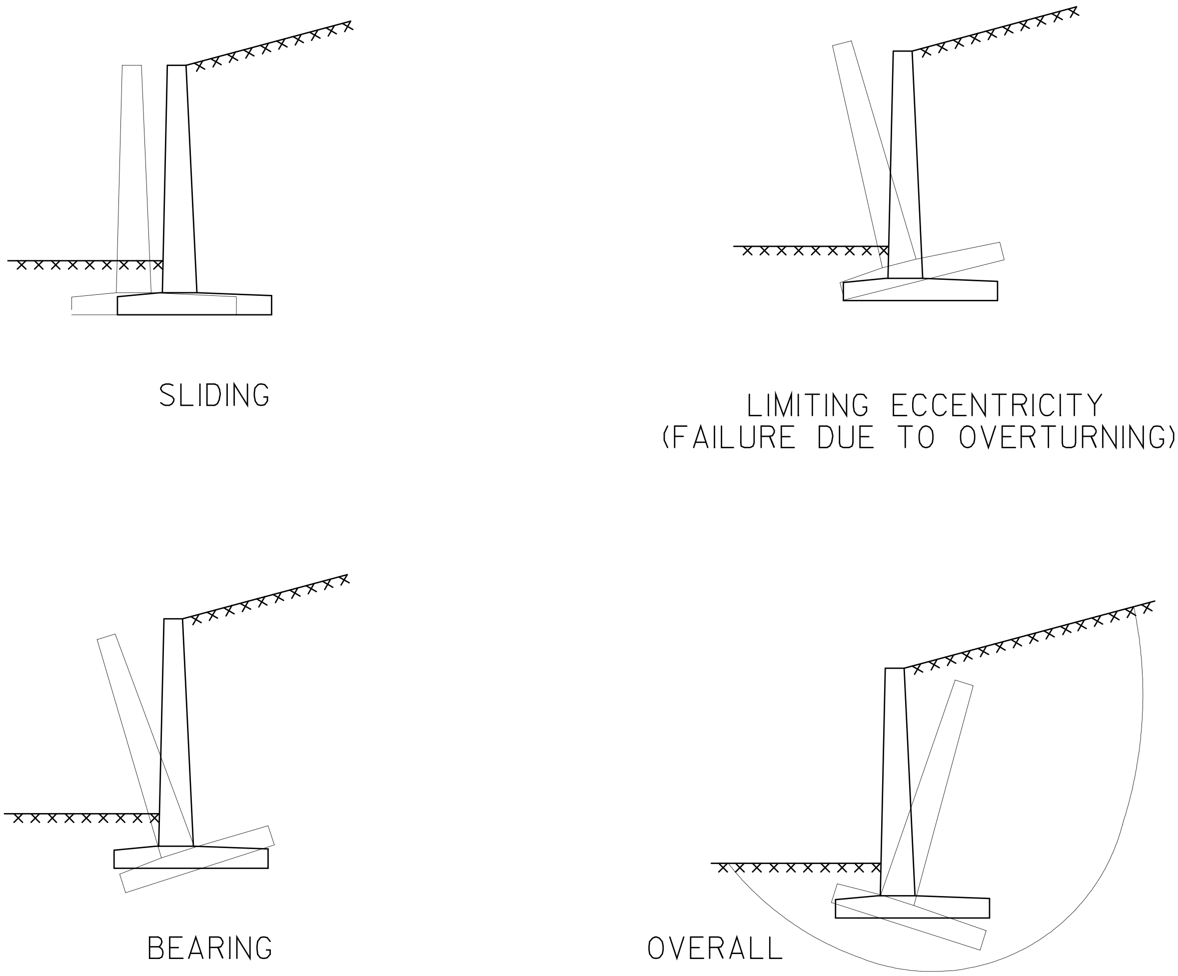


Figure 14.5-1
CIP Semi-Gravity Wall Failure Mechanism

14.5.2.1 Design Steps

The general design steps discussed in 14.4.1 shall be followed for the wall design. These steps as applicable for CIP cantilever walls are summarized below.

1. Establish project requirements including wall height, geometry and wall location as discussed in 14.1 of this chapter.

2. Perform Geotechnical investigation and testing

3. Develop soil strength parameters

4. Determine preliminary sizing for external stability evaluation

5. Determine applicable unfactored or nominal loads

6. Evaluate factored loads for all appropriate limit states

7. Perform stability check to evaluate bearing resistance, eccentricity, and sliding as part of external stability

8. Estimate wall settlement and lateral wall movement to meet guidelines stated in Table 14.4-3.

9. Check overall stability and revise design, if necessary, by repeating steps 4 to 8.

It is assumed that steps 1, 2 and 3 have been performed prior to starting the design process.

14.5.3 Preliminary Sizing

A preliminary design can be performed using the following guideline.

1. The wall height and alignment shall be selected in accordance with the preliminary plan preparation process discussed in 14.1.

2. Preliminary CIP wall design may assume a stem top width of 12 inches. Stem thickness at the bottom is based on load requirements and/or batter. The front batter of the stem should be set at ¼ inch per foot for stem heights up to 28 feet. For stem heights from 16 feet to 26 feet inclusive, the back face batter shall be a minimum of ½ inch per foot, and for stem heights of 28 ft maximum and greater, the back face shall be ¾ inch per foot per stability requirements.

3. Minimum Footing thickness for stem heights equal to or less than 10 ft shall be 1.5 ft and 2.0 ft when the stem height exceeds 10 ft or when piles are used.

4. The base of the footing shall be placed below the frost line, or 4 feet below the finished ground line. Selection of shallow footing or deep foundation shall be based on the geotechnical investigation, which should be performed in accordance with guidelines presented in Chapter 11 - Foundation Support.

5. The final footing embedment shall be based on wall stability requirements including bearing resistance, wall settlement limitations, external stability, internal stability and overall stability requirements.

6. If the finished ground line is on a grade, the bottom of footings may be sloped to a maximum grade of 12 percent. If the grade exceeds 12 percent, place the footings level and use steps.

The designer has the option to vary the values of each wall component discussed in steps 2 to 6 above, depending on site requirements and to achieve economy. See Figure 14.5-2 for initial wall sizing guidance.

Where

ΣV = Summation of vertical forces

B = Base width

e = Eccentricity as shown in Figure 14.5-3 and Figure 14.5-4

If the resultant is outside the middle one-third of the wall base, then the vertical stress shall be computed using:

$$\sigma_{V\,\max} = \left(\frac{2\sum V}{3\left(\frac{B}{2} - e\right)} \right)$$

$\sigma_{vmin} = 0$

The computed vertical stress shall be compared with factored bearing resistance in accordance with the **LRFD [10.6.3.1]** using following equation:

$q_r = \phi_b q_n > \sigma_v$

Where:

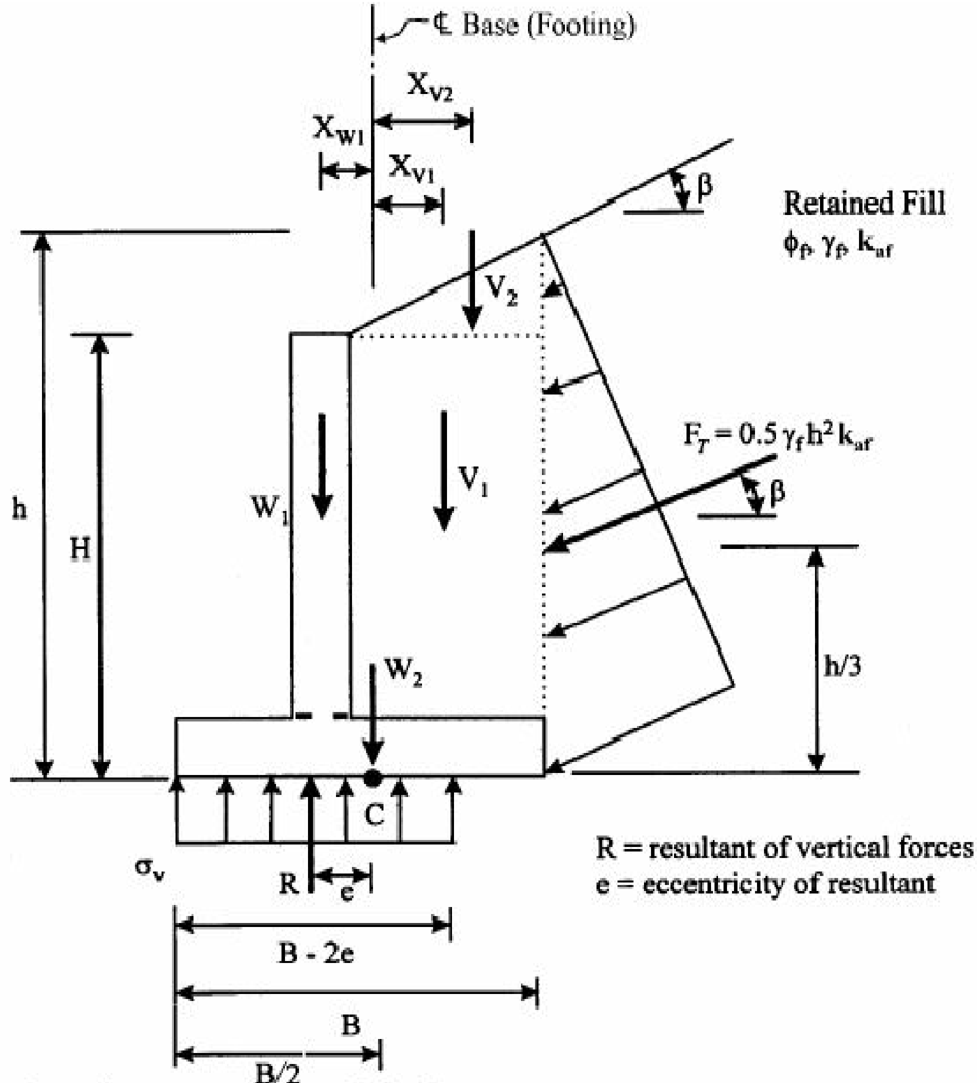
q_r = Factored bearing resistance

q_n = Nominal bearing resistance computed using **LRFD [10.6.3.1.2-a]**

σ_v = Vertical stress

B = Base width

e = Eccentricity as shown in Figure 14.5-3 and Figure 14.5-4



Summing moments about Point C:

$$e = \frac{\left(F_T Cos\beta\right)h/3 - \left(F_T \sin\beta\right)B/2 - V_1X_{V1} - V_2X_{V2} + W_1X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 11.6.3.2-1 Bearing Stress Criteria for Conventional Wall Foundations on Soil.

Figure 14.5-3
Loading Diagram and Bearing Stress Criteria for CIP Cantilever Walls on Soil
(source *AASHTO LRFD*)

14.5.10 Summary of Design Requirements

1. Stability Check

    a. Strength I and Extreme Event II limit states

    - Eccentricity
    - Bearing Stress
    - Sliding

    b. Service I limit states

    - Overall Stability
    - Settlement

2. Foundation Design Parameters

    Use values provided by Geotechnical analysis

3. Concrete Design Data

    - f'c = 3500 psi
    - fy = 60,000 psi

4. Retained Soil

    - Unit weight = 120 lb/ft^3
    - Angle of internal friction - use value provided by Geotechnical analysis

5. Soil Pressure Theory

    - Coulomb theory for short heels or Rankine theory for long heels at the discretion of the designer.

6. Surcharge Load

    - Traffic live load surcharge = 2 feet = 240 lb/ft^2
    - If no traffic surcharge, use 100 lb/ft^2

7. Load Factors

Group	γ_{DC}	γ_{EV}	γ_{LSv}	γ_{LSh}	γ_{EH}	γ_{CT}	Probable use
Strength I-a	0.90	1.00	1.75	1.75	1.50		Sliding, eccentricity
Strength I-b	1.25	1.35	1.75	1.75	1.50		Bearing /wall strength
Extreme II-a	0.90	1.00	-	-	-	1.00	Sliding, eccentricity
Extreme II-b	1.25	1.35	-	-	-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	1.00		Global/settlement/wall crack control

Table 14.5-8
Load Factor Summary for CIP Walls

8. Bearing Resistance Factors

For Standard Penetration test (SPT) and Footing on soils and Rock

- ϕ_b = 0.55 **LRFD Table [11.5.6-1]**

9. Sliding Resistance Factors

For SPT test and Footing on soils and Rock

- ϕ_τ = 1.0 **LRFD Table [11.5.6-1]**
- ϕ_{ep} = 0.5 **LRFD Table [10.5.5.2.2-1]**

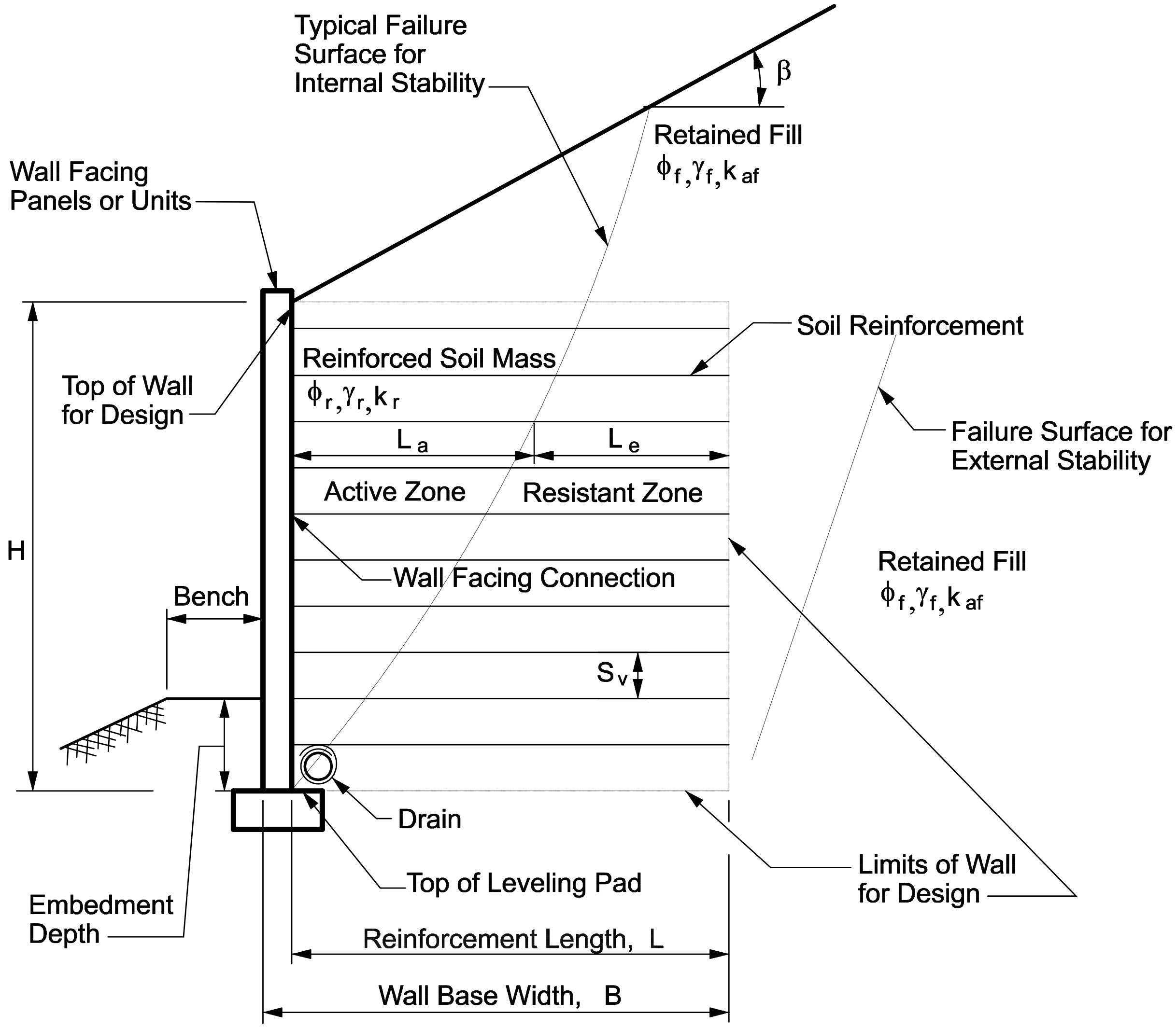


Figure 14.6-1
Structural Components of MSE Walls

14.6.2.1 Reinforced Earthfill Zone

The reinforced backfill to be used to construct the MSE wall shall meet the criteria in the wall specifications. The backfill shall be free from organics, or other deleterious material. It shall not contain foundry sand, bottom ash, blast furnace slag, or other potentially corrosive material. It shall meet the electrochemical criteria given in Table 14.6-1.

Reinforcement Material	Property	Criteria
Metallic	Resistivity	> 3000 ohm cm/H
Metallic	Chlorides	<100 ppm
Metallic	Sulfates	< 200 ppm
Metallic/Geosynthetic	pH	3.5<pH <9
Metallic/Geosynthetic	pH	4.5<pH <10

Table 14.6-1
Electrochemical Properties of Reinforced Fill MSE Walls

An angle of internal friction of 30 degrees and unit weight of 120 pcf shall be used for the stability analyses as stated in 14.4.6. If it is desired to use an angle of internal friction greater than 30 degrees, it shall be determined by the standard Direct Shear Test, AASHTO T-236, on the portion finer than the No. 10 sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T-99 (with the appropriate correction for coarse particles) at optimum moisture content. The maximum allowable angle of internal friction with testing, is 36 degrees. No testing is required for backfills where 80% of aggregates are greater than ¾-inch.

14.6.2.2 Reinforcement:

Soil reinforcement can be either metallic (strips or bar grids like welded wire fabric) or non-metallic including geo-textile and geogrids made from polyester, polypropylene, or high density polyethylene. Metallic reinforcements are also known as inextensible reinforcement and the non-metallic as extensible. Inextensible reinforcement deforms less than the compacted soil infill used in MSE walls, whereas extensible reinforcement deforms more than compacted soil infill

The metallic or inextensible reinforcement is mild steel, and usually galvanized or epoxy coated. Three types of steel reinforcement are typically used:

Steel Strips: The steel strip type reinforcement is mostly used with segmental concrete facings. Commercially available strips are ribbed top and bottom, 2 to 4 inch wide and 1/8 to 5/32 inch thick.

Steel grids: Welded wire steel grids using two to six W7.5 to W24 longitudinal wires spaced either at 6 or 8 inches. The transverse wire may vary from W11 to W20 and are spaced from 9 to 24 inches apart.

Welded wire mesh: Welded wire meshes spaced at 2 by 2 inch of thinner steel wire can also be used.

The galvanized steel reinforcement that is used for soil reinforcement is oversized in cross sectional areas to account for the corrosion that occurs during the life of the structure and the resulting loss of section. The net section remaining after corrosion at the end of the design service life is used to check design requirements

The non-metallic or extensible reinforcement includes the following:

Geogrids: The geogrids are mostly used with modular block walls.

Geotextile Reinforcement: High strength geotextile can be used principally with wrap-around and temporary wall construction.

Corrosion of the wall anchors that connect the soil reinforcement to the wall face must also be accounted for in the design.

14.6.2.3 Facing Elements

The types of facings element used in the different MSE walls mainly control aesthetics, provide protection against backfill sloughing and erosion, and may provide a drainage path in certain cases. A combination of different wall facings and reinforcement provide a choice of selecting an MSE wall which can be used for several different functions.

Major facing types are:

- Segmental precast concrete panels
- Dry cast or wet cast modular blocks
- Full height pre-cast concrete panels
- Cast-in-place concrete facing
- Geotextile-reinforced wrapped face
- Geosynthetic /Geogrid facings
- Welded wire grids

Segmental Precast Concrete Panels

Segmental precast concrete panels include small panels (<30 sq ft) to larger (>30 sq ft) with a minimum thickness of 5-½ inches and are of a square, rectangular, cruciform, diamond, or hexagonal geometry. The geometric pattern of the joints and the smooth uniform surface finish of the factory provided precast panels give an aesthetically pleasing appearance. Segmental precast concrete panels are proprietary wall components.

Wall panels are available in a plain concrete finish or numerous form liner finishes and textures. An exposed aggregate finish is also available along with earth tone colors. Although color can be obtained by adding additives to the concrete mix it is more desirable to obtain color by applying concrete stain and/or paint at the job site. Aesthetics do affect wall costs.

WisDOT requires that MSE walls utilize precast concrete panels when supporting traffic live loads which are in close proximity to the wall. Panels are also allowed as components of an abutment structure. Either steel strips or welded wire fabric is allowed for soil reinforcement when precast concrete panels are used as facing of the MSE wall system. Smaller panels shall be used for cases where radius of curvature of the wall is less than 50 feet. Contact Structures Design Section for approval on case by case basis. In general, MSE wall structures with panel type facings shall be limited to wall heights of 33 feet.

Concrete Modular Blocks Facings

Concrete modular block retaining walls are constructed from modular blocks typically weighing from 40 to 100 pounds each, although blocks over 200 pounds are rarely used. Nominal front to back width ranges between 8 to 24 inches. Modular blocks are available in a large variety of facial textures and colors providing a variety of aesthetic appearances. The shape of the blocks usually allows the walls to be built along a curve, either concave or convex. The blocks or units are dry stacked meaning mortar or grout is not used to bond the units together except for the top two layers. Figure 14.6-2 shows various types of blocks available commercially. Figure 14.6-3 shows a typical modular block MSE wall system along with other wall components.

Concrete modular blocks are proprietary wall component systems. Each proprietary system has its own unique method of locking the units together to resist the horizontal shear forces that develop. Fiberglass pins, stainless steel pins, glass filled nylon clips and mechanical interlocking surfaces are some of the methods utilized. Any pins or hardware must be manufactured from corrosion resistant materials.

During construction of these systems, the voids are filled with granular material such as crushed stone or gravel. Most of the systems have a built in or automatic set-back (incline angle of face to the vertical) which is different for each proprietary system. Blocks used on WisDOT projects must be of one piece construction. A minimum weight per block or depth of block (distance measured perpendicular to wall face) is not specified on WisDOT projects. The minimum thickness allowed of the front face is 4 inches (measured perpendicular from the front face to inside voids greater than 4 square inches). Also the minimum allowed thickness of any other portions of the block (interior walls or exterior tabs, etc.) is 2 inches.

It is WisDOT policy to design modular block MSE walls for a maximum height of 22 ft (measured from the top of the leveling pad to the top of the wall). Most modular block MSE walls systems use geogrids as reinforcement.

Where:

ΣM_V = Summation of Resisting moment due to vertical earth pressure

ΣM_H = Summation of Moments due to Horizontal Loads

ΣV = Summation of Vertical Loads

For eccentricity to be considered acceptable, the calculated location of the resultant vertical force (based on factored loads) should be within the middle one-half of the base width for soil foundations (i.e., e_{max} = B/4) and middle three-fourths of the base width for rock foundations (i.e., e_{max} = 3B/8). Therefore, for each load group, e must be less than e_{max}. If e is greater than e_{max}, a longer length of reinforcement is required. The CDR for eccentricity should be greater than 1.

CDR = e_{max}/e > 1

14.6.3.5.4 Bearing Resistance

The bearing resistance check shall be performed in accordance with **LRFD [11.10.5.4]**. Provisions of **LRFD [10.6.3.1]** and **LRFD [10.6.3.1]** shall apply**.** Because of the flexibility of MSE walls, an equivalent uniform base pressure shall be assumed. Effect of live load surcharge shall be added, where applicable, because it increases the load on the foundation. Vertical stress, $\sigma_{v,}$ shall be computed using following equation.

The bearing resistance computation requires:

Base Pressure $(\sigma_v) = \dfrac{\sum V}{B - 2e}$

σ_v = Vertical pressure

ΣV = Sum of all vertical forces

B = Reinforcement length

e = Eccentricity = B/2 – X_0

X_0 = $(\Sigma M_R - \Sigma\ M_H)/\Sigma V$

ΣM_V = Total resisting moments

$\Sigma\ M_H$ = Total driving moments

The nominal bearing resistance, q_n, shall be computed using methods for spread footings. The appropriate value for the resistance factor shall be selected from **LRFD Table [10.5.5.2.2.1]**.

The computed vertical stress, σ_v, shall be compared with factored bearing resistance, q_r in accordance with the **LRFD [11.5.6-1]** and a Capacity Demand Ratio, CDR, shall be calculated using the following equation:

$$q_r = \phi_b\ q_n \geq \sigma_v$$

Where:

q_r	=	Factored bearing resistance
q_n	=	Nominal bearing resistance computed using **LRFD [10.6.3.1.2a-1]**
ϕ_b	=	0.65 using **LRFD Table [11.5.6.1]**
CDR	=	q_r/σ_v >1.0

14.6.3.6 Vertical and Lateral Movement

Excessive MSE wall foundation settlement can result in damage to the wall facing, coping, traffic barrier, bridge superstructure, bridge end panel, pavement, and/or other settlement-sensitive elements supported on or near the wall.

Techniques to reduce damage from post-construction settlements and deformations may include full-height vertical sliding joints through the rigid wall facing elements and appurtenances, and/or ground improvement or reinforcement techniques. Staged preload/surcharge construction using onsite materials or imported fills may also be used.

Settlement shall be computed using the procedures outlined in 14.4.7.2 and the allowable limit settlement guidelines in 14.4.7.2.1 and in accordance with **LRFD [11.10.4]** and **LRFD [10.6.2.4].** Differential settlement from the front face to the back of the wall shall be evaluated, as appropriate.

For MSE walls with rigid facing concrete panels, slip joints of 0.75 inch width can be provided to control differential settlement as per **LRFD [Table C11.104.4-1]**.

14.6.3.7 Overall Stability

Overall Stability shall be performed in accordance with **LRFD [11.10.4.3]**. Provision of **LRFD [11.6.2.3]** shall also apply. Overall and compound stability of complex MSE wall system shall also be investigated, especially where the wall is located on sloping or soft ground where overall stability may be inadequate. Compound external stability is the responsibility of the contractor/wall supplier. The long term strength of each backfill reinforcement layer intersected by the failure surface should be considered as restoring forces in the limit equilibrium slope stability analysis. Figure 14.6-4 shows failure surfaces generated during overall or compound stability evaluation.

The correction factor, α, depends primarily upon the strain softening of compacted granular material , and the extensibility, and the length of the reinforcement. Typical value is given in Table 14.6-2.

Reinforcement Type	α
All steel reinforcement	1.0
Geogrids	0.8
Geotextiles	0.6

Table 14.6-2
Typical values of α
(Source **LRFD [Table 11.10.6.3.2-1]**)

The pullout friction factor, F*, can be obtained accurately from laboratory pullout tests performed with specific material to be used on the project. Alternating, lower bound default values can be used from the laboratory or field pull out test performed in the specific back fill to be used on the project.

As shown in Figure 14.6-5, the total length of reinforcement (L) required for the internal stability is computed as below

$$L = L_e + L_a$$

Where:

L_e = Length of reinforcement in the resistance zone

L_a = Remainder length of reinforcement

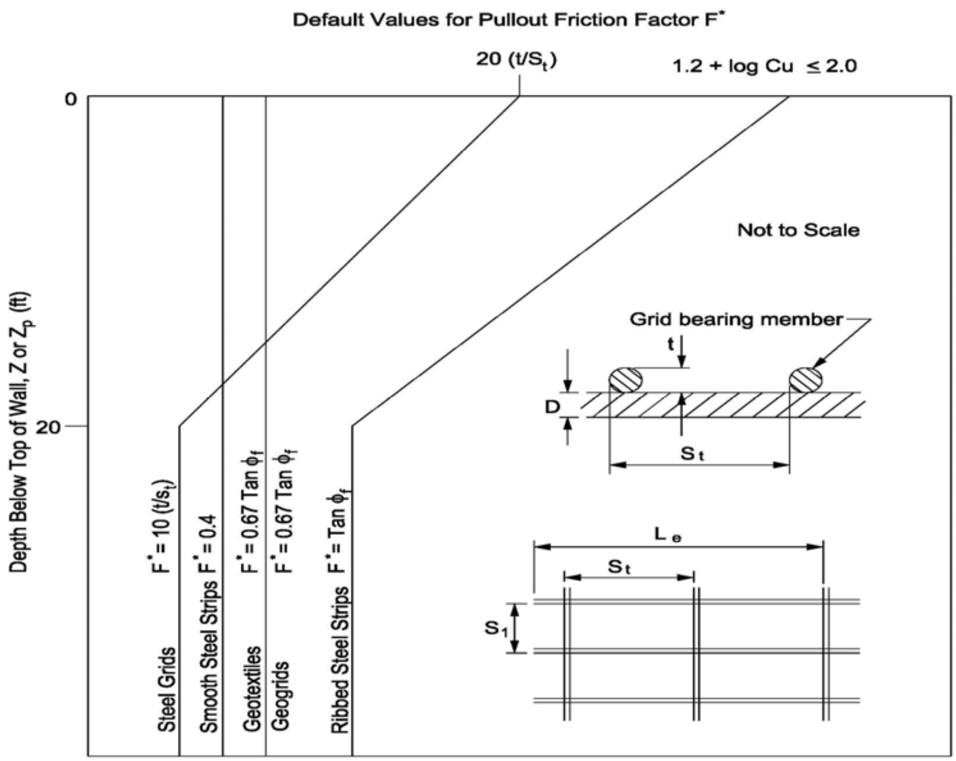


Figure 14.6-8
Typical Values of F*
(Source: **LRFD Figure [11.10.6.3.2-1]**)

14.6.3.8.6 Reinforced Design Strength

The maximum factored tensile stress (T_{MAX}) in each reinforcement layer as determined in 14.6.3.8.4 is compared to the long term reinforcement design strength computed in accordance with **LRFD [11.10.6.4.1]** as:

$$T_{MAX} \leq \phi T_{al} R_C$$

Where

ϕ = Resistance factor for tensile resistance

R_c = Reinforcement coverage ratio

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Application	One Design Lane Loaded	Two or More Design Lanes Loaded
Moment in Interior Girder – **LRFD [Table 4.6.2.2.2b-1]**		
	$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.1}$	$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.1}$
	For N_b = 3, use the lesser of the values obtained from the equations above with N_b = 3 or the lever rule.	
Shear in Interior Girder – **LRFD [Table 4.6.2.2.3a-1]**		
	$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$
	For N_b = 3, use the lever rule.	
Moment in Exterior Girder – **LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.2d-1]**		
	Use lever rule	$g=e\cdot g_{\text{int erior}}$ $e=0.77+\frac{d_e}{9.1}$
		For N_b = 3, use the lesser of the value obtained from the equation above with N_b = 3 or the lever rule.
Shear in Exterior Girder – **LRFD [C4.6.2.2.2d]** and **LRFD [Table 4.6.2.2.3b-1]**		
	Use lever rule	$g=e\cdot g_{\text{int erior}}$ $e=0.6+\frac{d_e}{10}$
		For N_b = 3, use the lever rule.
Moment Reduction for Skew – **LRFD [Table 4.6.2.2.2e-1]** (not applicable for WisDOT)		
Shear Correction for Skew – **LRFD [Table 4.6.2.2.3c-1]**		

Table 17.2-7

Commonly Used Live Load Distribution Factors for Girder Structures

WisDOT exception to AASHTO:

The rigid cross-section requirement specified in **LRFD [4.6.2.2.2d]** shall not be applied when calculating the distribution factors for exterior girders.

WisDOT exception to AASHTO:

For skewed bridges, WisDOT does not apply skew correction factors for moment reduction, as specified in **LRFD [Table 4.6.2.2.2e-1]**.

WisDOT policy item:

For skewed bridges, WisDOT applies the skew correction factor for shear, as specified in **LRFD [Table 4.6.2.2.3c-1]**, to the *entire span* for *all girders* in a multi-girder bridge.

The following variables are used in Table 17.2-7:

S	=	Spacing of beams (feet)
L	=	Span length (feet)
t_s	=	Depth of concrete slab (inches)
K_g	=	Longitudinal stiffness parameter (inches^4)
N_b	=	Number of beams or girders
g	=	Distribution factor
e	=	Correction factor for distribution
d_e	=	Distance from the exterior web of exterior beam to the interior edge of curb or traffic barrier (feet)

For shear due to live load, in addition to the equations presented in Table 17.2-7, a skew correction factor must be applied in accordance with **LRFD [4.6.2.2.3c-1]**. The skew correction factor equation for shear in girder bridges is as follows:

$$1.0 + 0.20\left(\frac{12.0 L t_s^{3}}{K_g}\right)^{0.3} \tan\theta$$

Where:

L	=	Span length (feet)
t_s	=	Depth of concrete slab (inches)
K_g	=	Longitudinal stiffness parameter (inches^4)
θ	=	Skew angle (degrees)

As a general rule of thumb, whenever the live load distribution factors are computed based on the equations presented in *AASHTO LRFD*, the multiple presence factor has already been considered and should not be applied by the engineer. However, when a sketch must be drawn to compute the live load distribution factor, the multiple presence factor must be applied to the computed distribution factor. An example of this principle is in the application of the lever rule.

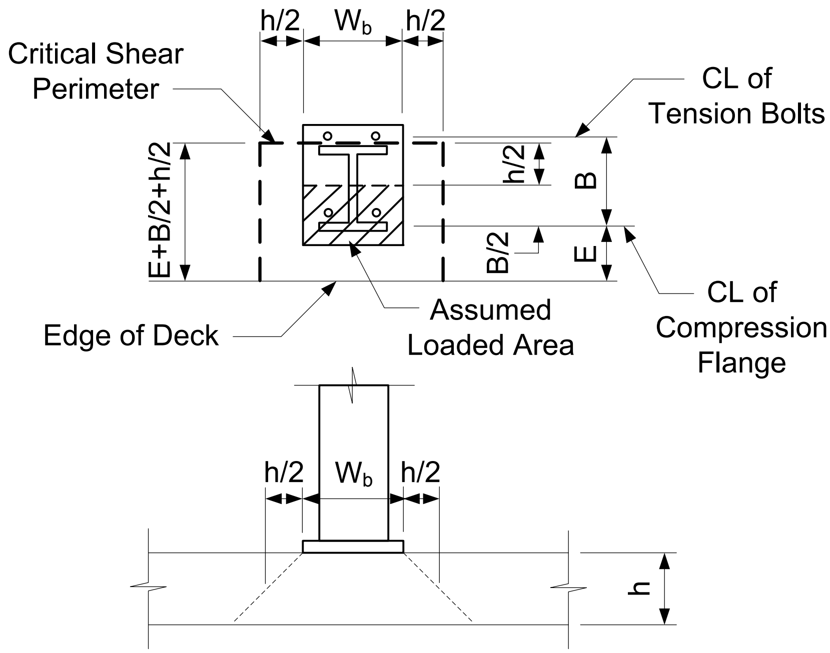

Figure 17.6-6

Assumed Load Distribution for Punching Shear

As used in Figure 17.6-6:

B	=	Distance between centroids of tensile and compressive stress resultants in post (inches)
E	=	Distance from edge of slab to centroid of compressive stress resultant in post (inches)
h	=	Depth of slab (inches)
W_b	=	Width of base plate (inches)

The design loads for Design Case 3 are dead and live loads, as illustrated in Figure 17.6-7.

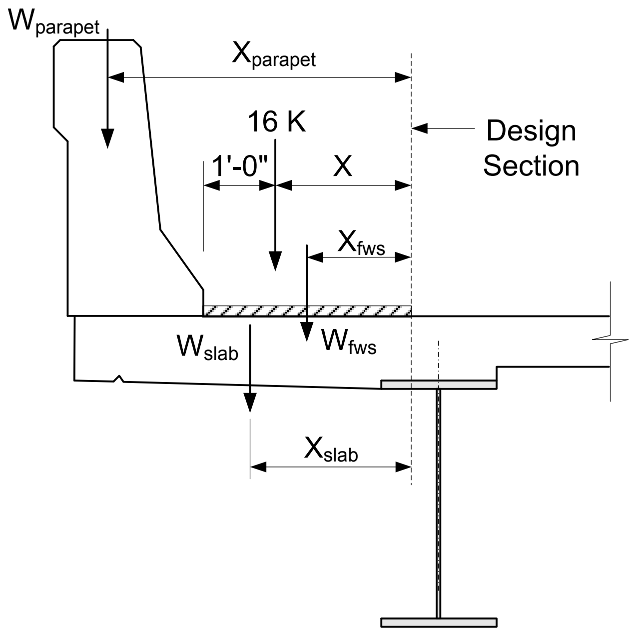

Figure 17.6-7
Design Case 3

As presented in **LRFD [Table 4.6.2.1.3-1]**, the equivalent strip (in the longitudinal direction), in units of inches, for live load on an overhang for Design Case 3 is:

Equivalent strip = 45.0 + 10.0X

Where:

X = Distance from load to point of support (feet), as illustrated in Figure 17.6-7

The multiple presence factor of 1.20 for one lane loaded and a dynamic load allowance of 33% should be applied, and the moment due to live load and dynamic load allowance is then computed.

Based on the computations for the three design cases, the controlling design case and design location are identified. The factored design moment is used to compute the required reinforcing steel. Cracking in the overhang must be checked for the service limit state in accordance with **LRFD [5.7.3.4]**. The controlling overhang reinforcement for cantilever deck slabs is shown in Table 17.6-2 and Table 17.6-3 for single slope and sloped face concrete parapets, and in Table 17.6-4 and Table 17.6-5 for steel railing Type "M". Type "W" railing is no longer allowed on girder structures.

If the area of reinforcing steel required in the overhang is greater than the area of reinforcing steel provided by the standard transverse reinforcement over the interior girders, it shall be placed as detailed in Figure 17.6-8.

17.6.1 Rail Loading for Slab Structures

For concrete slab superstructures, the designer is required to consider the rail loading and provide adequate transverse reinforcing steel, accordingly. The top transverse slab reinforcement for both concrete parapet and steel railing type "M" or "W" are shown on the Standard Details.

17.6.2 WisDOT Overhang Design Practices

WisDOT policy item:

Current design practice in Wisconsin limits the standard slab overhang length to 3'-7", measured from the centerline of the exterior girder to the edge of the slab. A 4'-0" overhang is allowed for some wide flange prestressed concrete girders (54W", 72W", 82W"). A 4'-6" overhang may be used where a curved roadway is placed on straight girders at the discretion of the designer. The total overhang when a cantilevered sidewalk is used is limited to 5'-0", measured from the centerline of the exterior girder to the edge of the sidewalk. A minimum of 6" from the edge of the top flange to the edge of the deck should be provided, with 9" preferred.

The overhang length has been limited to prevent rotation of the girder and bending of the girder web during construction caused by the eccentric load from the cantilevered forming brackets. The upper portion of these brackets attaches to the girder top flange, and the lower portion bears against the girder web. If the girder rotates or the web bends at the bracket bearing point, the end of the bracket will move downward because of bracket rotation. If the rails supporting the paving machine are located near the end of the bracket, the paving machine will move downward more than the girder and the anticipated profile grade line will not be achieved. Factors affecting girder rotation are diaphragm spacing, stiffness, connections and girder torsional stiffness. Factors affecting web bending are stiffener spacing and web thickness. Do not place a note or detail on the plan for exterior girder bracing required by the contractor as this is covered by the specs.

In the following tables, the slab thickness, "t", is the slab thickness between interior girders. The area of steel shown in the following tables is the controlling value from Design Case 1, 2 or 3. The value shown is the larger area of steel required at the front face of the barrier or at the design section. Girder Type 1 shall include steel girders and the following prestressed girders; 28-inch, 36-inch, 36W-inch, 45W-inch. Prestressed girders used for rehabilitation projects, 45-inch, 54-inch and 70-inch, shall also be considered to be Girder Type 1. Girder Type 2 shall include the following prestressed girders; 54W-inch, 72W-inch and 82W-inch.

If the area of reinforcing steel required in the overhang is greater than the area of reinforcing steel provided by the standard transverse reinforcement over the interior girders, reinforcement must be added to satisfy the overhang design requirements. The amount of reinforcement that must be added in the overhang is the amount required to satisfy the overhang design requirement minus the amount provided by the standard transverse reinforcement over the interior girders. This additional reinforcement should be carried for the bar development length past the exterior girder centerline. The reinforcement shall be placed as detailed in Figure 17.6-8. Use either a number 4 or 5 bar to satisfy this requirement. The additional bar shall be placed at one or two times the standard transverse bar spacing as required.

Effective Overhang (Feet)	Deck Thickness Between Girders, "t" (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.75	0.749	0.690	0.640	0.597	0.562	0.529	0.514
2.00	0.747	0.690	0.643	0.603	0.568	0.536	0.510
2.25	0.766	0.706	0.655	0.612	0.576	0.545	0.517
2.50	0.781	0.718	0.666	0.622	0.584	0.551	0.523
2.75	0.793	0.728	0.675	0.629	0.591	0.557	0.527
3.00	0.805	0.738	0.682	0.636	0.596	0.562	0.532
3.25	0.815	0.745	0.688	0.642	0.601	0.566	0.535
3.50	0.824	0.752	0.694	0.646	0.605	0.569	0.538
3.75	0.849	0.761	0.700	0.650	0.608	0.572	0.541
4.00	0.959	0.862	0.785	0.688	0.636	0.590	0.544

Table 17.6-2
Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Single Slope or Sloped Face Concrete Parapets --- Girder Type 1

Effective Overhang (Feet)	Deck Thickness Between Girders, "t" (Inches)						
	8	8.5	9	9.5	10	10.5	11
1.25	0.749	0.691	0.644	0.603	0.568	0.537	0.511
1.5	0.761	0.700	0.649	0.607	0.570	0.537	0.510
1.75	0.761	0.700	0.649	0.606	0.570	0.537	0.510
2	0.761	0.700	0.649	0.606	0.570	0.537	0.510
2.25	0.740	0.681	0.632	0.591	0.555	0.547	0.526
2.5	0.735	0.678	0.629	0.588	0.553	0.559	0.541
2.75	0.732	0.674	0.626	0.586	0.550	0.549	0.557
3	0.730	0.673	0.626	0.584	0.550	0.539	0.553
3.25	0.729	0.672	0.624	0.584	0.549	0.528	0.543

Table 17.6-3
Reinforcing Steel (in^2/ft) for Cantilever Deck Slabs with Single Slope or Sloped Face Concrete Parapets --- Girder Type 2

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28" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	54	60
6’-6”	54	58
7’-0”	52	56
7’-6”	50	54
8’-0”	50	54
8’-6”	48	52
9’-0”	48	50
9’-6”	46	50
10’-0”	44	48
10’-6”	44	48
11’-0”	42	46
11’-6”	42	46
12’-0”	42	44

36" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	72	78
6’-6”	70	76
7’-0”	70	74
7’-6”	68	72
8’-0”	66	70
8’-6”	64	68
9’-0”	62	68
9’-6”	60	64
10’-0”	60	64
10’-6”	58	62
11’-0”	58	60
11’-6”	50	60
12’-0”	48	58

36W" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	98	104
6’-6”	96	102
7’-0”	94	100
7’-6”	92	98
8’-0”	88	96
8’-6”	86	94
9’-0”	84	92
9’-6”	82	88
10’-0”	80	86
10’-6”	78	84
11’-0”	76	82
11’-6”	74	80
12’-0”	72	78

45W" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	116	124
6’-6”	114	122
7’-0”	112	118
7’-6”	108	116
8’-0”	106	114
8’-6”	102	110
9’-0”	100	108
9’-6”	98	104
10’-0”	94	102
10’-6”	94	100
11’-0”	90	98
11’-6”	88	96
12’-0”	86	92

Table 19.3-1
Maximum Span Length vs. Girder Spacing

54W" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	132	140
6’-6”	128	136
7’-0”	126	134
7’-6”	124	132
8’-0”	122	130
8’-6”	118	126
9’-0”	116	124
9’-6”	114	120
10’-0”	112	118
10’-6”	110	116
11’-0”	108	114
11’-6”	106	112
12’-0”	102	110

72W" Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	160*⊗	168*⊗
6’-6”	158*	168*⊗
7’-0”	154*	164*⊗
7’-6”	152	162*⊗
8’-0”	150	158*
8’-6”	146	154*
9’-0”	144	152
9’-6”	140	148
10’-0”	138	146
10’-6”	136	144
11’-0”	132	140
11’-6”	130	138
12’-0”	128	134

82W” Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	174*⊗	184*⊗
6’-6”	170*⊗	180*⊗
7’-0”	168*⊗	178*⊗
7’-6”	164*⊗	174*⊗
8’-0”	162*⊗	172*⊗
8’-6”	158*	168*⊗
9’-0”	156*	164*⊗
9’-6”	152	162*⊗
10’-0”	150	158*
10’-6”	148	156*
11’-0”	144	152
11’-6”	142	150
12’-0”	140	146

Table 19.3-2
Maximum Span Length vs. Girder Spacing

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

lift location based on f'_{ci}. A note should be placed on the girder details sheet to reflect that the girder was analyzed for a potential lift at the 1/10 point.

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

19.3.8.1 Pretensioned I-Girder Standard Strand Patterns

The standard strand patterns presented in the Standard Details were developed to eliminate some of the trial and error involved in the strand pattern selection process. These standard strand patterns should be used whenever possible, with a straight strand arrangement preferred over a draped strand arrangement. The designer is responsible for ensuring that the selected strand pattern meets all LRFD requirements.

Section 19.3.3 discusses the key parts of the design procedure, and how to effectively use the standard strand patterns along with Table 19.3-1 and Table 19.3-2.

The amount of drape allowed is controlled by the girder size and the 2" clearance from center of strand to top of girder. See the appropriate Standard Girder Details for guidance on draping.

19.3.9 Precast, Prestressed Slab and Box Sections Post-Tensioned Transversely

These sections may be used for skews up to 30° with the transverse post-tensioning ducts placed along the skew. Skews over 30° are not recommended, but if absolutely required the transverse post-tensioning ducts should be placed perpendicular to the prestressed sections. Also for skews over 30° a more refined method of analysis should be used such as a two-dimensional grid analysis or a finite element analysis.

WisDOT policy item:

These sections may be used on all off-system bridges and for on-system bridges with ADT ≤ 300. The maximum skew for these types of bridges shall be 30°. Variations to these requirements require prior written approval by the WisDOT BOS Development Section.

Details for transverse post-tensioning are shown on the Standard for Prestressed Slab and Box Girder Sections as well as Prestressed Slab and Box Girder Details. Post-tensioning ducts shall be placed along the skew. Each post-tensioning duct contains three ½" diameter strands which produce a total post-tensioning force per duct of 86.7 kips. Post-tensioning ducts are located at each end of the beams (slab or box section), at the ¼ point and the ¾ point of the beams, and at 10-foot maximum spacing between the ¼ and ¾ points.

Precast slab or box sections are subject to high chloride ion exposure because of longitudinal cracking that sometimes occurs between the boxes or from drainage on the fascia girders when an open steel railing system is used. To reduce permeability the concrete mix is required to contain fly ash as stated in 503.2.2 of the Standard Specifications except that the amount of portland cement replaced with fly ash shall be in a range of 20 to 25 percent. Also an entrained air content of 8% air, +/- 1.5% is required.

When these sections are in contact with water for 5-year flood events or less, the sections must be cast solid for long term durability. When these sections are in contact with water for the 100-year flood event or less, any voids in the section must be cast with a non-water-absorbing material.

Table 19.3-3 provides approximate span limitations for pretensioned slab and box sections as a function of section depth and roadway width. It also gives the section properties associated with these members. Criteria for developing these tables are shown below Table 19.3-3.

19.3.9.1 Available Slab and Box Sections and Maximum Span Lengths

Precasters have forms available to make six precast pre-stressed box sections ranging in depth from 12" to 42". Each section can be made in widths of 36" and 48", but 48" is more efficient and is the preferred width. Typical box section information is shown in the Standard Details.

Table 19.3-3 shows available section depths and section properties and maximum span length. The maximum span lengths are based on 21, 0.6" diameter strands (18 for 12" section) and HL93 loading. All sections have voids except the 12" deep section.

	Section No.	Section Depth (inches)	Section Area, A, (in^2)	Moment of Inertia, I, (in^4)	Section Modulus, S, (in^3)	Torsional Inertia, J, (in^4)	Max. Span (ft)
3'-0" Section Width	1	12	432	5,184	864	16,796	24
	2	17	435	14,036	1,651	26,110	44
	3	21	475	25,012	2,382	43,161	52
	4	27	548	49,417	3,660	73,829	64
	5	33	608	83,434	5,057	109,192	72
	6	42	698	155,317	7,396	168,668	82
4'-0" Section Width	1	12	576	6,912	1,152	23,419	24
	2	17	555	18,606	2,189	40,962	44
	3	21	595	32,942	3,137	68,601	50
	4	27	668	64,187	4,755	119,322	64
	5	33	728	107,204	6,497	178,965	76
	6	42	818	196,637	9,364	281,253	92

Table 19.3-3
Box Girder Section Properties and Maximum Span Length

Table based on:

- HL93 loading and AASHTO LRFD Bridge Design Specifications
- Interior girder of a simple span structure
- f'_c = 5 ksi and f'_{ci} = 4.25 ksi
- 0.5 or 0.6" dia., low relaxation prestressing strands at $0.75f'_s$
- f'_s = 270.0 ksi
- 2" min. concrete overlay (which doesn't contribute to stiffness of section)
- Assumed M rail weight distributed evenly to all girder sections
- 30° skew used to compute diaphragm weight
- 1 ½" of mortar between sections
- Post-tensioning diaphragms located as stated in 19.3.9
- 30'-0" minimum clear bridge width (ten 3'-0" sections, eight 4'-0" sections)

19.3.9.2 Overlays

There are three types of overlays that can be used on these structures.

1. Concrete Overlay, Grade E or C
2. Asphaltic Overlay with Waterproofing Membrane
3. Modified Mix Asphalt

19.3.9.3 Mortar Between Precast, Prestressed Slab and Box Sections

These sections are typically set 1 ½" apart and the space between sections is filled with a mortar mix prior to post-tensioning the sections transversely. Post-tensioning is not allowed until the mortar has cured for at least 48 hours.

19.4 Field Adjustments of Pretensioning Force

When strands are tensioned in open or unheated areas during cold weather they are subject to loss due to change in temperature. This loss can be compensated for by noting the change in temperature of the strands between tensioning and initial set of the concrete. For purposes of uniformity the strand temperature at initial concrete set is taken as 80°F.

Minor changes in temperature have negligible effects on the prestress force, therefore only at strand temperatures of 50°F and lower are increases in the tensioning force made.

Since plan prestress forces are based on 75% of the ultimate for low relaxation strands it is necessary to utilize the AASHTO allowable of temporarily overstressing up to 80% to provide for the losses associated with fabrication.

The following example outlines these losses and shows the elongation computations which are used in conjunction with jack pressure gages to control the tensioning of the strands.

Computation for Field Adjustment of Prestress Force

Known:

22 - 1/2", 7 wire low relaxation strands, A_{ps} = 0.1531 in^2

P_{pj} = 710.2 kips (jacking force from plan)

T_1 = 40°F (air temperature at strand tensioning)

T_2 = 80°F (concrete temperature at initial set)

L = 300' = 3,600" (distance from anchorage to reference point)

L_1 = 240' = 2,880" (length of cast segment)

E_p = 29,000 ksi (of prestressing tendons, sample tested from each spool)

C = 0.0000065 (coefficient of thermal expansion for steel, per degree F.)

COMPUTE:

jacking force per strand = P_{pj} = 710.2/22 = 32.3 kips

DL_1 = PL/AE = 32.3 x 3600/(0.1531 x 29,000) = 26.1"

Initial Load of 1.5 Kips to set the strands

DL_2 = 1.5 x 3600/(0.1531 x 29000) = 1.22"

DL_3 = Slippage in Strand Anchors = 0.45" (Past Experience)

DL_4 = Movement in Anchoring Abutments = 0.25" (Past Experience)

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30.1 Crash Tested Bridge Railings

All bridge railings must have passed the crash tests as recommended in the NCHRP report 350 for Bridge Railings. In order to use railings other than Bridge Office Standard railing details, the railings must conform to crash tested rails which are available from the FHWA office. Any railings that are not crash tested must be reviewed by FHWA when they are used on bridge, culvert, retaining wall, etc.

Railings must meet the criteria for TL-3 or greater to be used on all roadways. Railings meeting TL-2 criteria may be used on roadways where the speed is 45 mph or less.

30.2 Railing Application

WisDOT policy item:

Bridge plans utilizing concrete parapets for any 2012 PS&E shall be designed with "LF" (or "HF" or "51F") parapets. Bridge plans utilizing concrete parapets for PS&E after 2012 shall be designed with "32SS" (or "36SS", "42SS", "56SS") parapets. (An exception could be made if final design is far enough along that changing to the "SS" shape would be burdensome.) If it is known that roadway concrete barrier single slope (CBSS) is going to be used on a project with a 2012 PS&E, single slope parapet "__SS" should be used for the bridges.

The designation for railing types are shown on the Standards. Standard railing details are generally employed as follows:

1. The single slope parapet "32SS" is preferred on state and interstate highway bridges (see "WisDOT policy item" above) except for some limited short span structures. The "36SS" and "42SS" parapets are used where there is high truck traffic and curved horizontal alignment creating more potential for overtopping the parapet, or if roadway concrete barrier single slope (CBSS) adjoins the bridge. Single slope parapet "56SS" is only used if 56" CBSS adjoins the bridge. These parapets meet crash test criteria for TL-4.

   A "SS" or solid parapet is preferred on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The "LF" parapet is preferred on state and interstate highway bridges (see "WisDOT policy item" above) except for some limited short span structures. The "HF" parapet is used where there is high truck traffic and curved horizontal alignment creating more potential for overtopping the parapet. These parapets meet crash test criteria for TL-4.

3. Type "H" aluminum or steel railing is used on top of either the Vertical Face Parapet "A" or sloped face parapets ("__SS", "LF" or "HF") when required for pedestrians and/or bicyclists. Both situations, with or without the "H" rail, meet the criteria for TL-4. For a design speed greater than 45 mph, the sloped face parapet is recommended. If the structure has a sidewalk on one side only, the sloped face parapet is used on the side opposite of the sidewalk.

4. Type "F" steel railing is not allowed on the National Highway System (NHS). Type "F" railing may be used on non-NHS roadways with a design speed of 45 mph or less. This railing facilitates drainage and snow removal but is usually more expensive than the Sloped Face Parapet if drains are not required at the ends of the bridge. In order to meet AASHTO Specifications three or more posts are attached to the Type "F" railing. May be used when TL-2 criteria is required.

5. Type "M" steel railing is used on state maintained bridges where the Region requests an open railing. It is similar to the Type "F" but has a higher crash test rating. Used in place of the Type "W" rail on girder type structures. Type "M" railing is not, however, allowed on prestressed box girder bridges. Meets criteria for TL-4.

6. Type "W" railings may be used on all functional classes of Wisconsin highway structures. Generally, Type "W" railing is considered when the highway approach requires standard beam guard and if the structure is 80 feet or less in length. Meets criteria for TL-3. The Type “W” rail shall only be used on concrete slab structures. The use of this railing on girder type structures shall be discontinued.

7. Aesthetic railings may be used if crash tested according to Section 30.1. The Texas style parapet, Type “TX”, has been crash tested but it is very expensive. Form liners to simulate the openings would reduce the cost of this parapet. Meets criteria for TL-2.

   The Standards show some Combination Railings, Type “C1” through “C6” that are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are at least 5” from the crash-tested rail face and do not present a snagging potential. Meets criteria for TL-2, but should only be used when the design speed is 45 mph or less, or the railing is protected by a barrier between the roadway and sidewalk.

8. The “51F” parapet may only be used on the median side (see “WisDOT policy item” above), when it provides a continuation of the approach 51 inch high median barrier.

9. The Type “PF” tubular railing is not allowed on the National Highway System (NHS). Type “PF” railing may be used on non-NHS roadways with a design speed of 45 mph or less. This railing is similar to the Type “F” railing with two main differences. The height of this rail meets the requirements for pedestrian facilities. This is a solid rail type that can be used on a grade separation structure. May be used when TL-2 criteria is required.

10. If a box culvert has beam guard railing across the structure, then the rail members shall have provisions for a Thrie Beam connection at the ends of the structure as shown on SDD 14 B 20 Standards. Railing is not required on box culverts if there is a clear zone as defined in *Facilities Development Manual 11-15-1*. Non-Traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a traffic barrier to shield the hazard or obstacle may be warranted. The barrier shall be provided only when it is cost effective as defined in *Facilities Development Manual Procedure 11-45-1*.

11. When the structure approach beam guard is extended across the box culvert; refer to Standard Detail, Box Culvert Details for additional information. The minimum dimension between end of box and face of guard rail provides an acceptable rail deflection to prevent a vehicle wheel from traversing over the end of the box culvert. In almost every case, the timber posts with offset blocks and standard beam guard are used. Type "W" railing may be used for maintenance and box culvert extensions to mitigate the effect of structure modifications.

12. Timber Railing as shown in the Standards is not allowed on the National Highway System (NHS). Timber Railing may be used on non – NHS roadways with a design speed of 45 mph or less. Meets criteria for TL-2.

13. Chain Link Fence and Ornamental Protective Screening, as shown in the Standards, may be attached to the top of concrete parapets (or directly to the deck if on a sidewalk separated from the roadway by a crashworthy barrier). Ornamental Protective Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a barrier between the roadway and sidewalk. Chain Link Fence can be used for any design speed.

See the *Facilities Development Manual 11-40-1* for additional railing application requirements. See 11-35-1, Table 1.1 for requirements on when barrier wall separation between roadway and sidewalk is required.

30.3 Design Details

1. Provide for expansion movement in tubular railings where expansion devices or concrete parapet deflection joints exist on the structure plan details. The tubular railing splice should be located over the joint at its midspan and made continuous with a movable internal sleeve. On conventional structures where expansion joints are likely to occur at the abutments only, if tubular railing is employed, the posts may be placed at equal increments providing that no post is nearer than 2 feet from deflection joints in the parapet at the piers.

2. Epoxy coated bars are required for all concrete parapets, curbs, medians, and sidewalks.

3. Refer to Standard Detail, Vertical Face Parapet "A" for detailing concrete parapet or median deflection joints. These joints are used because of previous experience with transverse deck cracking beneath the parapet joints.

4. Horizontal cracking occurred near the top of some concrete parapets which were slip formed. Similar cracking has not occurred on parapets cast in forms. Therefore, slip forming of bridge parapets should not be allowed.

5. Detail erection joints at the one-sixth panel point for Type "F" railing. This location will insure primarily a shear transfer at the railing splices. For beam guard type railing, locate the expansion splice at a post or on either side of the expansion joint.

6. On skewed bridges where the length from the rail post to the first guard rail posts exceeds 3 feet, employ the following detail: Extend the railing to the back face of the abutment. Bolt a plate to the back of the rails right before the rail bend. In case of vehicle impact, this detail will cause the rails to act as a unit in preventing vehicle wheel snagging.

7. Note the AASHTO Specification for a maximum opening of 6 inches on lower rail elements.

8. Sidewalks - If there is a parapet between the roadway and a sidewalk and the roadway side of the parapet is more than 11'-0" from the exterior edge of deck, the sidewalk width must be 10'-0" clear between barriers. Access must be provided to the sidewalk for the "snooper truck" to inspect the underside of the bridge. The boom extension on most trucks does not exceed 11'-0" so provision must be made to get the truck closer to the edge.

9. The height of curbs for sidewalks is usually 6 inches. This height is more desirable than higher heights with regards to safety because it is less likely to vault vehicles.

30.4 Utilities

The maximum allowable conduits that can be placed in "__SS", "HF" or "LF" parapets are shown in the following sketches ("LF" only shown). Junction (Pull) boxes can only be used with 2 inch diameter conduit. The maximum length of 3 inch conduit is 190 feet, as no boxes are allowed.

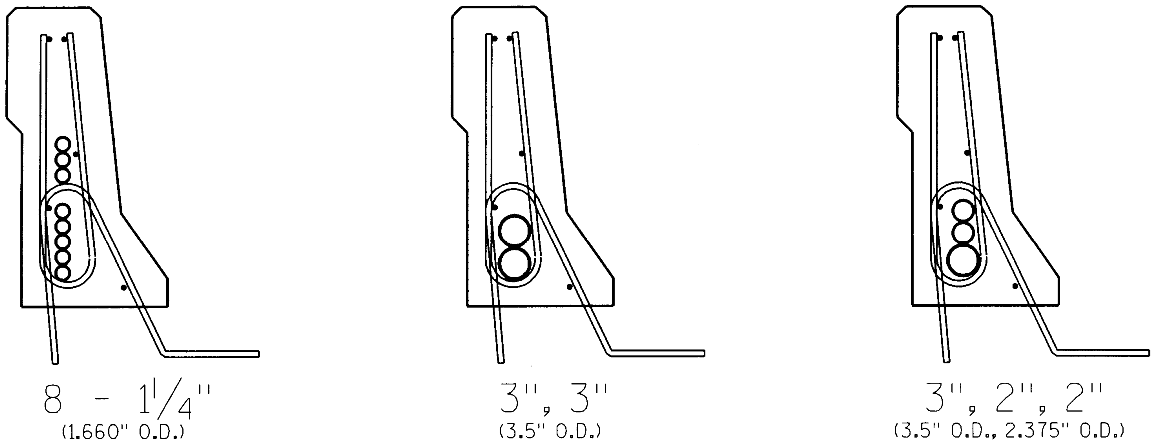

Figure 30.4-1
Maximum Allowable Conduits in "__SS", "HF" and "LF" Parapets

When light poles are mounted on top of parapets and the design speed exceeds 45 mph the light pole must be located behind the back edge of the parapet. The poles should also be placed over the piers unless there is an expansion joint. Place 4 feet away if this is the case.

FDM 9-25-5 addresses whether a bench mark disk should be set on a structure. Structures are not usually preferred due to possible elevation changes from various causes. WisDOT has discontinued the statewide practice of furnishing a disk and requiring it to be placed on a structure. WisDOT Region Offices may continue to provide a bench mark for the contract to be set. Consult the Region Office to determine if a bench mark should be included in the plan set.

30.5 Protective Screening

Protective screening is a special type fence constructed on the sides of an overpass to discourage and/or prevent people from dropping or throwing objects onto vehicles passing underneath the structure. Protective screening is generally chain link type fencing attached to steel posts mounted on top of a vertical parapet or on a sidewalk surface. The top of the protective screening may be curved inward toward the structure, if mounted on a parapet and on a sidewalk, to prevent objects from being thrown off the overpass structure. Aesthetics is enhanced by using a colored protective screening which can be coordinated with the color of the structure. See Chapter 37 for screening details.

Examples of situations that warrant consideration of protective screening are:

1. If there is a history of or instances of objects being dropped or thrown from an existing overpass.

2. For all new overpasses if there have been instances at other existing overpasses in the area.

3. On overpasses near a school, playground, residential area or any other location where the overpass may be used by children who are not accompanied by an adult.

In addition, all pedestrian overpasses should have protective screening on both sides.

When protective screening is warranted, the minimum design should require screening on the side of the structure with sidewalk. Designers can call for protective screening on sides without sidewalks if those sides are readily accessible to pedestrians.

Designers should insure that where protective screening is called for, it does not interfere with sight distances between the overpass and any ramps connecting it with the road below. This is especially important on cloverleaf and partial cloverleaf type interchanges.

Protective screening is not always warranted. An example of when it may not be warranted is on an overpass without sidewalks where pedestrians do not have safe or convenient access to either side because of high traffic volumes and/or the number of traffic lanes that must be crossed.

Occasionally access to light poles behind protective screening is required or the screening may need repair. To gain access, attach fence stretchers to the fencing and remove one vertical wire by threading or cutting. The vertical wire may be cut without using fence stretchers. To repair, attach fence stretchers and thread a vertical wire in place of the one removed by either reusing the one in place or using a new one.

Fence repair would follow this same process except the damaged fencing would be removed and replaced with new fencing.

30.6 Medians

The typical height of any required median curb is 6 inches. This will prevent normal crossovers and reduce vaulting on low speed roadways without excessive dead load. The preferred median for structures is shown on the Standard for Median and Raised Sidewalk Details.

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36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of AASHTO LRFR does not cover rating of concrete box culverts. See 45.8 for values to place on the plans for inventory and operating rating factors.

WisDOT Policy Item:

Current WisDOT policy is to not rate box culverts. In the future, rating requirements will be introduced as AASHTO LRFR is updated to address box culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) equal to 1.5. See section 45.6 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans. The current version of AASHTO LRFR does not cover rating of concrete box culverts. See 45.8 for values to place on the plans for maximum (Wis-SPV) vehicle load.

36.6.6 Interior Wall Moment Steel

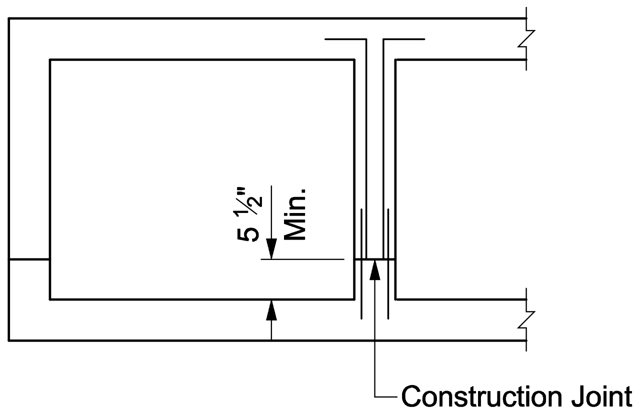

Figure 36.6-5
Layout of Interior Wall Steel

The area of steel is determined from the maximum moment at the top of the wall and the effective wall thickness. A minimum of #4 bars at 18 inches is supplied. Identical steel is provided at both faces of the wall. A 1 foot, 90 degree bend, is provided in the top slab with the horizontal portion being just below the negative moment steel. The dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed. When a haunch is provided, the construction joint is placed a distance above the bottom slab equal to the haunch depth plus 2 inches.

36.6.7 Distribution Reinforcement

Per **LRFD [5.14.4.1]**, transverse distribution reinforcement is not required for culverts where the depth of fill exceeds 2.0 feet.

Per **LRFD [12.11.2.1]**, provide distribution reinforcement for culverts with less than or equal to 2 feet of fill in accordance with **LRFD [9.7.3.2]**, which states that reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows (for primary reinforcement parallel to traffic):

$$\text{Percentage} = \frac{100}{\sqrt{S}} \leq 50\%$$

Where:

S = Effective span length (ft) (for slabs monolithic with walls, this distance is taken as the face-to-face distance per **LRFD [9.7.2.3]**)

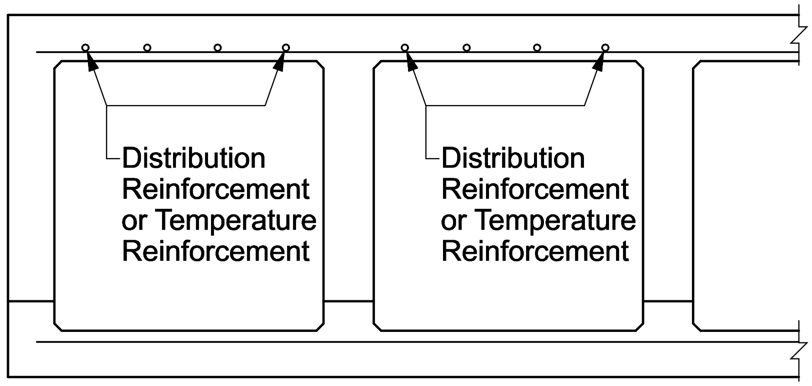

Figure 36.6-6
Layout of Distribution Steel

36.6.8 Temperature Reinforcement

Temperature reinforcement is required on all wall and slab faces in each direction that does not already have strength or distribution reinforcement. Per **LRFD [12.11.4.3.1]**, provide shrinkage and temperature reinforcement in walls and slabs in accordance with **LRFD [5.10.8]**, which states that the area of shrinkage and temperature steel per foot on each face and in each direction shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$

$$0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	Area of reinforcement in each direction and each face (in^2/ft)
b	=	Least width of component section (in.)
h	=	Least thickness of component section (in.)
f_y	=	Specified yield strength of reinforcing bars ≤ 75 (ksi)

Where the least dimension varies along the length of the component, multiple sections should be examined to represent the average condition at each section.

Temperature steel is always #4 bars at a maximum spacing of 18 inches. When the top slab has no fill on top use a minimum of #4 bars at 12 inch centers in both directions in the top of the top slab.

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spalling of concrete surfaces and exposure of reinforcing steel, or disintegration of masonry jointing.

- Dolphins - Where depth of water and other conditions are suitable, the driving of pile clusters may be considered. Such clusters have the piles lashed together with cable to promote integral action. The clusters should be flexible to be effective in absorbing impact through deflection.

- Cellular dolphins - May be filled with concrete, loose materials or materials suitable for grouting.

- Floating shear booms - Where the depth of water or other conditions precludes the consideration of dolphins or integral pier protection, floating sheer booms may be used. These are suitably shaped and positioned to protect the pier and are anchored to allow deflection and absorption of energy. Anchorage systems should allow for fluctuations in water level due to stream flow or tidal action.

- Hydraulic devices - Such as suspended cylinders engaging a mass of water to absorb or deflect the impact energy may be used under certain conditions of water depth or intensity of impact. Such cylinders may be suspended from independent caissons, booms projecting from the pier or other supports. Such devices are customarily most effective in locations subject to little fluctuations of water levels.

- Fender systems - Constructed using piling with horizontal wales, is a common means of protection where water depth is not excessive and severe impacts are not anticipated.

- Other types of various protective systems have been successfully used and may be considered by the Engineer. Criteria for the design of protective systems cannot be specified to be applicable to all situations. Investigation of local conditions is required in each case, the results of which may then be used to apply engineering judgment to arrive at a reasonable solution.

38.4 Overpass Structures

Highway overpass structures are placed when the incidences of train and vehicle crossings exceeds certain values specified in the *Facilities Development Manual* (*FDM*). The separation provides a safer environment for both trains and vehicles.

In preparing the preliminary plan which will be sent to the railroad company for review and approval several items of data must be determined.

- Track Profile - In order to maintain clearances under existing structures when the track was upgraded with new ballast, the railroad company did not change the track elevation under the structure causing a sag in the gradeline. The track profile would be raised with a new structure and the vertical clearance for the structure should consider this.

- Drainage - Hydraulic analysis is required if any excess drainage will occur along the rail line or into existing drainage structures. Deck drains shall not discharge onto railroad track beds.

- Horizontal Clearances - The railroad system is expanding just as the highway system. Contact the railroad company for information about adding another track or adding a switching yard under the proposed structure.

- Safety Barrier – The Commissioner of Railroads has determined that the Transportation Agency has authority to determine safety barriers according to their standards. The railroad overpass parapets should be designed the same as highway grade separation structures using solid parapets (Type “SS” or appropriate) and pedestrian fencing where required.

38.4.1 Preliminary Plan Preparation

Standard for Highway over Railroad Design Requirements shows the minimum dimensions for clearances and footing depths. These should be shown on the Preliminary Plan along with the following data.

- Milepost and Direction - Show the railroad milepost and the increasing direction.

- Structure Location - Show location of structure relative to railroad right of way. (Alternative is to submit Roadway Plan).

- Footings - Show all footing depths. Minimum depth from top of rail to top of footing is 6’-6”.

- Drainage Ditches - Show ditches and direction of flow.

- Utilities - Show all utilities that are near structure footings and proposed relocation is required.

- Crash Protection – See Standard for Highway over Railroad Design Requirements for crash protection requirements. On a structure widening a crashwall shall be added if the multi-columned pier is less than 20 feet from centerline of track. Site circumstances will determine whether a crashwall is needed when the distance is between 20 and 25 feet.

- Shoring - If shoring is required, use a General Note to indicate the location and limit.

38.4.2 Final Plans

The Final Plans must show all the approved Preliminary Plan data and be signed and/or sealed by a Registered Engineer.

38.4.3 Shoring

Railroad companies are particularly concerned about their track elevations. It is therefore very important that shoring is used where required and that it maintains track integrity.

38.4.4 Horizontal and Vertical Clearances

38.4.4.1 Horizontal Clearance

The distance from the centerline of track to the face of back slopes at the top of rail must not be greater than 20’-0” since federal funds are not eligible to participate in costs for providing greater distances unless required by site conditions. Minimum clearances to substructure units are determined based on site conditions and the character of the railroad line. Consideration must be given to the need for future tracks. Site specific track drainage requirements and possible need for an off-track roadway must also be considered.

38.4.4.2 Vertical Clearance

Section 192.31, Wisconsin Statutes requires 23’-0” vertical clearance above top of rail (ATR) for new construction or reconstruction, unless the Office of the Commissioner of Railroads approves less clearance. As a result, early coordination with the Railroads and Harbors Section is required.

Double stack containers at 20’-2” ATR are the highest equipment moving in restricted interchange on rail lines which have granted specific approval for their use. Allowing for tolerance, this equipment would not require more than 21’-0” ATR clearance. Railroad companies desire greater clearance for maintenance purposes and to provide allowance for possible future increases in equipment height.

38.4.4.3 Compensation for Curvature

Where a horizontal clearance obstruction is within 80 feet of curved track AREMA specifications call for lateral clearance increases as stated in *AREMA Manual* Chapter 28, Table 28-1-1.

38.4.4.4 Constructability

The minimum clearances discussed are to finished permanent work. Most railroad companies desire minimum temporary construction clearances to forms, falsework or track protection of 12'- 0" horizontal and 21'-0" vertical. The horizontal clearance provides room for a worker to walk along the side of a train and more than ample room for a train worker who may be required to ride on the side of a 10'-8" wide railroad car. Where piers are to be located close to tracks the type of footing to be used must be given careful consideration for constructability. The depth of falsework and forms for slab decks may also be limited by temporary vertical clearance requirements.

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- Combined distress area is less than 25%.

- May require crack sealing the following year and periodically thereafter.

* Note: Or another PC concrete product as approved by Structures Development and coordinated with the Region.

40.5.3 Maintenance Notes

- All concrete overlays crack immediately. If the cracks in the deck are not sealed periodically, the rate of deterioration can increase rapidly.

- AC overlays with a waterproofing membrane can also be used on new decks or older decks that are in good condition as preventive maintenance.

40.5.4 Special Considerations

On continuous concrete slab bridges with extensive spalling in the negative moment area, not more than 1/3 of the top bar steel should be exposed if the bar ends are not anchored. This is to maintain the continuity of the continuous spans and should be stated on the final structure plans.

If more than 1/3 of the steel is exposed, either the centers of adjacent spans must be shored or only longitudinally overlay 1/3 of the bridge at a time.

40.5.5 Railings and Parapets

The top of the overlay should not go above the 3-inch vertical portion of a concrete parapet. Additionally, overlays increase vehicle lean over sloped face parapets resulting in vehicles on bridges with higher ADT and/or speed having an increased likelihood of impact with lights/obstructions on top of, or behind, the parapet.

Sub-standard railings and parapets should be improved. An example of such a sub-standard barrier would be a curb with a railing or parapet on top. Contact the Bureau of Structures Development section to discuss solutions.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. Refer to the criteria in 40.3 of this chapter for additional 3R project considerations. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges eligible for deck replacements:

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

Table 40.6-1
Condition Requirements for Deck Replacements

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

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

WisDOT policy item:

Please contact the Bureau of Structures Development Section if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating greater than HS18, but less than HS20.

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

See the *Facilities Development Manual* for anchorage/offset requirements for temporary barrier used in staged construction.

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

For prestressed girder deck replacements, only use intermediate steel diaphragms in locations of removed intermediate concrete diaphragms (i.e. don't add intermediate lines of diaphragms).

40.7 Rehabilitation Girder Sections

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

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

The 45", 54", and 70" girders in Chapter 40 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 – Superstructures - 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
- Haunch height = 2" or 2 ½"
- Required f'_c girder at initial prestress < 6,800 psi

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

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

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

Table 40.7-1
Maximum Span Length vs. Girder Spacing

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

40.8 Widenings

Deck widenings, except on the Interstate, are attached to the existing decks if they are structurally sound and the remaining width is more than 50 percent of the total new width. Also, reference is made to the criteria in 40.3 of this Chapter for additional 3R project considerations. If the existing deck is over 20 percent surface delaminated or spalled, the existing deck shall be replaced. For all deck widenings on Interstate Highway bridges, the total deck should be replaced in order that total deck life is equal and costs are likely to be less when considering future traffic control. Evaluate the cost of traffic control for deck widenings on other highway bridges. The total deck should be replaced in these cases where the life-cycle cost difference is minimal if future maintenance costs are substantially reduced.

The design details must provide a means of moment and shear transfer through the joint between the new and existing portions of the deck. Lapped reinforcing bars shall have adequate development length and are preferable to doweled bars. The reinforcing laps must be securely tied or the bars joined by mechanical methods. When practical, detail lapped rebar splices. Mechanical splice couplers or threaded rebar couples are expensive and should only be detailed when required. Generally, shear transfer is more than sufficient without a keyway. Bridge Maintenance Engineers have observed that if the existing bar steel is uncoated, newly lapped coated bars will accelerate the uncoated bar steel deterioration rate.

When widening a deck on a prestressed girder bridge, if practical, use the latest standard shape (e.g. 54W” rather than 54”). Spacing the new girder(s) to maintain comparable bridge stiffness is desirable. *The girders used for widenings may be the latest Chapter 19 sections designed to LRFD or the sections from Chapter 40 designed LFD, or LRFD with non-prestressed reinforcement as detailed in the Standard Details.*

For multi-columned piers, consider the cap connection to the existing cap as pinned and design the new portion to the latest LRFD criteria. The new column(s) are not required to meet AASHTO 3.6.5 (400 kip loading) as a widening is considered rehabilitation. *It is intended to provide standard details in the Bridge Manual for a crash barrier that could, at the option of the Region, be used to strengthen and provide motorists protection for existing piers, including widenings.*

Abutments shall be widened to current LRFD criteria as well as the Standard for Abutment Widening.

For foundation support, use the most current method available.

All elements of the widening shall be designed to current LRFD criteria. Railings and parapets placed on the new widened section shall be up to current design and safety standards. For the non-widened side, substandard railings and parapets should be improved. Contact the Bureau of Structures Development section to discuss solutions.

For prestressed girder widenings, only use intermediate steel diaphragms in-line with existing intermediate diaphragms (i.e. don’t add intermediate lines of diaphragms).

40.9 Superstructure Replacements/Moved Girders (with Widening)

When steel girder bridges have girder spacings of 3' or less and require expensive redecking or deck widening; consider replacing the superstructure with a concrete slab. Approval is required from BOS for all Superstructure replacement projects. Also, additional costs will occur if bearings, floor drains, expansion joints, railings, or other structural elements require replacement.

When replacing a deck on a steel girder bridge, the cost of painting and structural rehab shall be considered to determine whether a deck or superstructure replacement is more cost effective (provided the substructure is sufficient to support the loading).

Evaluate the existing piers using current LRFD criteria. If an existing multi-columned pier has 3 or more columns, the 400 kip vehicular impact loading need not be considered if the pier is adjacent to a roadway with a design speed ≤ 40 mph. If the design speed is 45 mph or 50 mph, the 400 kip vehicular impact loading need not be considered if a minimum of "vehicle protection" is provided as per FDM 11-35-1. For design speeds > 50 mph, all criteria as per 13.4.10 must be met.

For abutments, evaluate the piles, or bearing capacity of the ground if on spread footings, utilizing Service I loading. The abutment body should be evaluated using Strength I loading.

The superstructure shall be design to current LRFD criteria.

40.10 Replacement of Impacted Girders

When designing a replacement project for girders that have been damaged by vehicular impact, replace the girder in-kind using the latest details. Either HS loading or HL-93 loading may be used if originally designed with non-LRFD methods. Consider using the latest deck details, especially with regard to overhang bar steel.

For the parapet or railing, the designer should match the existing.

40.11 New Bridge Adjacent to Existing Bridge

For a new bridge being built adjacent to an existing structure, the design of the new structure shall be to current LRFD criteria for the superstructure and abutment.

The pier design shall be to current LRFD criteria, including the 400 kip impact load for the new bridge. It is not required to strengthen or protect the existing adjacent pier for the 400 kip impact load. However, it would be prudent to discuss with the Region the best course of action. If the Region wants to provide crash protection, it may be desirable to provide TL-5 barrier/crash wall protection for both structures, thus eliminating the need to design the new pier for the 400 kip impact load. The Region may also opt to provide typical barrier protection (< TL-5) to both sets of piers, in which case the design engineer would still be required to design the new pier for the 400 kip impact load. This last option is less expensive than providing TL-5 barrier to both structures. Aesthetics are also a consideration in the above choices.

40.12 Timber Abutments

The use of timber abutments shall be limited to rehabilitation or widening of off-system structures.

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

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

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

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

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the special provisions.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30 for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. If shear connectors are on the existing top flanges, additional shear connectors are not to be added as welding to the top flange may be detrimental. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Existing steel expansion devices shall be modified or replaced with Watertight Expansion Devices as shown in Bridge Manual Chapter 28. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6’ or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide down hill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

40.14 Superstructure Inspection

40.14.1 Prestressed Girders

On occasion prestressed concrete girders are damaged during transport, placement, or as a result of vehicle impact. Damage inspection and assessment procedures are necessary to determine the need for traffic restrictions, temporary falsework for safety and/or strength. Three predominant assessment areas are damage to prestressing strands, damage to concrete, and remaining structural integrity.

Where damage to a girder results in any significant loss of concrete section, an engineering analysis should be made. This analysis should include stress calculations for the damaged girder with comparison to the original design stresses. These calculations will show the loss of strength from the damage.

Assessment of damage based on the loss of one or more prestressing strands or loss of prestress force is given as reason for restriction or replacement of girders. Some of the more common damages are as follows with the recommended maintenance action. However, an engineering assessment should be made on all cases.

1. If cracking and spalling are limited to the lower flange, patching is usually performed. This assessment should be based on calculations that may allow repair.

2. If cracking continues from flange into web, the girder is normally replaced. Findings indicate that sometimes repair-in-place may be the preferred decision.

3. Termini of cracks are marked, and if the cracks continue to grow the girder may be replaced. This is a good method to determine the effect of loads actually being carried.

4. When large areas of concrete are affected, or when concrete within stirrups is fractured, replace. At times it may be more appropriate to repair-in-place than replace.

Repair and/or replacement decisions are based on structural integrity. Load capacity is by far the most important rationale for selection of repair methods. Service load capacity needs to be calculated if repair-in-place is contemplated.

Evaluations of criteria for assessment of damage related to the two predominant areas associated with structural integrity are the following:

1. A structural analysis is made to determine the stresses in a damaged girder on the basis of the actual loads the girder will carry. This technique may result in a girder that has less capacity than the original, but can still safely carry the actual loads. This assessment results in consideration of possible repair-in-place methods which normally are cost effective.

   or

2. A structural analysis is made to determine the load capacity and rating of the girder. If the capacity and rating of the girder is less than provided by the original design, the girder shall be replaced. This assessment will provide a girder equal to the original design, but precludes possible repair-in-place methods that are normally less costly.

Location and size of all spalled and unsound concrete areas shall be recorded. Location, length, and width of all visible cracks shall be documented. All damage to prestress strands and reinforcing steel shall be reported. Location of hold-down devices in the girder shall be shown in relation to the damage. Horizontal and vertical misalignment along the length of the girder, and at points of localized damage, shall be reported. (These measurements might best be made by string-lining). Growth of cracks shall be monitored to determine that the cracked section has closed before extending to the web.

Critical damage is damage to concrete and/or the reinforcing elements of prestressed concrete girders such as:

1. Cracks extend across the bottom flange and/or in the web directly above the bottom flange. (This indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset is measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.

4. Vertical misalignment in excess of the normal allowable.

5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface. (This indicates permanent deformation of stirrups).

40.14.2 Steel Beams

These are three alternate methods of repairing damaged steel beams. They are:

1. Replace the total beam.

2. Replace a section of the beam.

3. Straighten the beam in-place by heating and jacking.

The first alternate would involve removing the concrete deck over the damaged beam, remove the damaged section and weld in a new piece; then reconstruct the deck slab and railing over the new girder. Falsework support is required at the locations where the beams are cut and probably in the adjacent span due to an unbalanced condition.

The second alternate involves cutting out a section of the beam after placing the necessary supporting members. The support is placed using calibrated jacks. The section is cut out as determined by the damage. A new section plus any vertical stiffeners and section of cover plates would be welded in. This involves butt welds on both the flange and web. The welding of the web is difficult due to minor misalignments to start with plus the tendency of thin plates to move from the heat of welding.

The third alternate of heating and jacking the in-place beam to straighten it is a difficult procedure but can be done by personnel familiar and knowledgeable of the process. It is important to maintain heat control under 1300°F maximum. Use an optical pyrometer to determine heat temperature. There is no specified tolerance for the straightened member. The process is deemed satisfactory when a reasonable alignment is obtained.

Based on the three alternates available, the estimated costs involved and the resultant restoration of the beam to perform its load carrying capacity, heat straightening is a viable option in many cases.

The structural engineer who will be responsible for plan preparation should field review the site with the Regional Bridge Maintenance Engineer.

40.15 Substructure Inspection

The inspection of substructure components may reveal deteriorating concrete in areas exposed to de-icing chemicals from roadway drainage or concrete disintegration where exposed in splash zones or stream flows. Footings and pilings exposed due to erosion and undermining could result in loss of bearing capacity and/or section.

Abutment and pier concrete reuse may require core tests to determine the quality and strength of the concrete. Reuse of steel pile sections will require checking the remaining allowable load carrying capacity. Steel piling should be checked immediately below the splash zone or water line for deterioration and possible loss of section. High section loss has occurred in some areas due to corrosion from bacterial attack at 3 to 6 feet below the water line. Bearing capacities of existing footings and pilings may have to be recomputed in order to determine if superstructure loading can be safely carried.

Timber substructure components may exhibit deterioration due to fungus decay, abrasion wear and weathering. Also, physical damage may be caused by vermin attacks, chemicals, fires, and collisions. Prior to reuse, timber backed abutments and pier bents shall be checked, by boring, for material and mechanical condition, section loss and structural adequacy.

40.15.1 Hammerhead Pier Rehabilitation

Pier caps and sometimes shafts of these piers have become spalled due to leaky joints in the deck. The spalling may be completely around some of the longitudinal bar steel destroying the bond. However, experience shows that the concrete usually remains sound under the bearing plates. There is no known reason for this except that maybe the compressive forces may prevent salt intrusion or counteract freeze thaw cycles.

If the longitudinal bars are full length, the bond in the ends insures integrity even though spalling may occur over the shaft. Corrective action is required as follows:

1. Place a watertight expansion joint in the deck.
2. Consider whether bearing replacement is required.
3. Analyze the type of cap repair required.
    a. Clean off spalled concrete and place new concrete.
    b. Analyze capacity of bars still bonded to see if unbonded bars are needed. Use ultimate strength analysis.
    c. Consider repair method for serious loss of bar steel capacity.
        i. Add 6” of cover to cap. Add additional bar steel. Grout in U shaped stirrups around bars using standard anchor techniques.

ii. Use steel plates and post-tensioning bars to place compression loads on both ends of cap. Cover exposed bars with concrete.

iii. Pour wing extension under pier caps beginning at base to take all loads in compression. This would alter pier shape.

d. Consider sloping top of pier to get better drainage.

e. Consider placing coating on pier top to resist water intrusion.

If the shaft is severely spalled, it will require a layer of concrete to be placed around it. Otherwise, patching is adequate. To add a layer of concrete:

1. Place anchor bars in existing solid concrete. Use expansion anchors or epoxy anchors as available.

2. Place wire mesh around shaft.

3. Place forms and pour concrete. 6” is minimum thickness.

40.15.2 Bearings

All steel bridge bearings should be replaced as shown in Bridge Manual Chapter 27. Compressive load and adhesion tests will be waived for steel laminated elastomeric bearings where these bearings are detailed to meet height requirements.

In general replace lubricated bronze bearings with teflon bearings. If only outside bearings are replaced, the difference in friction factors can be ignored. Where lubricated bronze bearings might be used, following is the design criteria.

For the expansion bearings, two additional plates are employed over fixed bearings, a top plate and a lubricated bronze plate. Current experience indicates that a stainless steel top plate reduces corrosion activity and is the recommended alternate to steel. The top plate is set on top of a lubricated bronze plate allowing expansion and contraction to occur. Laboratory testing of lubricated bronze plates indicated a maximum coefficient of friction varying from 8 to 14 percent for a loading of 200 kips. Current Office practice for steel girder Type “A” and prestressed girder expansion bearings of employing a 10 percent maximum and a 6 percent minimum friction value for design is in accordance to laboratory test results. For Type “A” bearing details refer to Standard Details.

40.16 Concrete Masonry Anchors for Rehabilitation

“Type S” and “Type L” concrete masonry anchors are used mostly for bridge rehabilitation projects and anchoring rail posts. One of the main differences between the two types of anchors is the duration of loading. It may be helpful to think of the “S” as Short-term loading and the “L” as Long-term loading. “Type L” anchors have greater embedment lengths to account for longer duration loading.

For both types of anchors the minimum pullout capacity specified on the plan is equal to (A_s x f_y), where f_y equals 60 ksi for rebar and 42 ksi for stainless steel bolts. The nominal resistance values shown in Table 40.16-1 and Table 40.16-2 are used in conjunction with the resistance factor, ϕ, for both tension and shear equal to 0.65. (*AASHTO* currently does not have resistance factors for anchors.) *It should be noted that WisDOT is currently evaluating adhesive anchored parapet replacements, which would be crash tested and consideration of the resistance factor would not be required*.

For all non-mechanical anchors, a two-part adhesive is either mixed and poured into a drilled hole or pumped into the hole by a dispenser which combines the two components at the nozzle just prior to entering the hole or within the hole. The hole must be properly cleaned and a sufficient amount of adhesive used so that the hole is completely filled with adhesive when the rebar or bolt is inserted.

"Type S" anchors are either mechanical wedge or adhesive anchors for installing studs, rebar, or bolts of a designated size. They are primarily used for anchoring bolts for attaching rail posts or other bolted objects and mostly smaller size rebars. “Type S” mechanical wedge anchors are seldom used for bridge rehabilitation. Because of creep, shrinkage and deterioration under load and freeze-thaw cycles, "Type S" adhesive anchors should not be used in situations where the rebar experiences a constant tension stress. When “Type S” anchors are used to anchor rebars, the rebar is listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS (Coated) Bridges”.

“Type L” anchors are adhesive anchors used to anchor rebars when the rebar is subject to continuous loading. “Type L” anchors of adequate length are capable of developing the tension strength of the bar for indefinite periods of time. When embedded a development length, rebars will develop the nominal tensile resistance value for sustained loading with a “Type L” anchor. “Type L” anchors are typically used for abutment and pier widenings, but may be used in other applications. For “Type L” anchors the rebar is listed in the “Bill of Bars” and paid for under the bid item “Bar Steel Reinforcement HS (Coated) Bridges”.

Usage Restrictions: Pier cap extensions for multi-columned piers require additional column(s) to be utilized. See Chapter 13 – Piers for structural modeling concepts regarding multi-columned piers. **Contact the Bureau of Structures if considering any extension of a hammerhead pier (without additional vertical support from an added column) or for vertical overhead installations.**

The manufacturer and product name of the “Type L” anchors and “Type S” adhesive anchors used by the contractor must be on the Department’s approved product list for “Concrete Masonry Anchors, Type L”.

Type S Anchor Size	Embedment Depth* in	Minimum Spacing in	Minimum Edge Distance in	Nominal Tensile Resistance kips	Resist. Factor ϕ	Factored Tensile Resistance kips
#4 or 1/2”	5	8	4	12	0.65	8
#5 or 5/8”	6	8	4	19	0.65	12
#6 or 3/4”	7	8	4	26	0.65	17
#7 or 7/8”	7	12	6	36	0.65	23
#8 or 1”	9	12	6	47	0.65	30
#9 or 1-1/8”	11	12	6	60	0.65	39

Table 40.16-1

Design Table for Concrete Masonry Anchors, Type S
(* Embedment for adhesive anchors, all bars. Mechanical anchor embedment by manufacturer. For anchors with f_y not equal to 60 ksi, adjust resistance accordingly)

Type L Anchor Size	Coated Rebar Embedment Depth ft-in	Uncoated Rebar Embedment Depth ft-in	Minimum Spacing in	Minimum Edge Distance in	Nominal Tensile Resistance kips	Resist. Factor ϕ	Factored Tensile Resistance kips
#4	1-0	1-0	8	4	12	0.65	8
#5	1-6	1-0	8	4	19	0.65	12
#6	1-10	1-3	8	4	26	0.65	17
#7	2-5	1-8	12	6	36	0.65	23
#8	3-3	2-2	12	6	47	0.65	30
#9	4-1	2-9	12	6	60	0.65	39

Table 40.16-2

Design Table for Concrete Masonry Anchors, Type L

The Factored Nominal Shear Resistance value equals the Factored Tensile Resistance multiplied by 0.66. This value is for 3.0 ksi concrete and above. If used in lower strength concrete, reduce the value by the ratios of the concrete moduli of rupture.

The Nominal and Factored Nominal Tensile Resistance values in the tables vary linearly with Embedment Depth. Use at least 80% of the Embedment Depths specified in the tables for Type S or Type L anchors.

The required minimum pullout capacity (as stated on the plans) is equal to the Nominal Tensile Resistance.

Typical notes for bridge plans (shown in all capital letters):

MASONRY ANCHORS TYPE S 5/8-INCH. MIN. PULLOUT CAPACITY OF 19 KIPS. FOR ADHESIVE ANCHORS, EMBED A MINIMUM OF 6" IN CONCRETE.

When using "Type S" anchors to anchor bolts or studs, the bolt, nut, and washer or the stud as detailed on the plans is included in the bid item.

For "Type S" anchors using rebar, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

MASONRY ANCHORS TYPE L NO. 5 BARS. MIN. PULLOUT CAPACITY OF 19 KIPS. EMBED A MINIMUM OF 1'-6" IN CONCRETE.

For "Type L" anchors, the rebar is listed in the "Bill of Bars" and paid for under the bid item "Bar Steel Reinforcement HS (Coated) Bridges".

It should be noted that AASHTO is considering adding specifications pertaining to concrete masonry anchors. This chapter will be updated once that information is available.

40.17 Plan Details

1. Excavation for Structures on Overlays

   There is considerable confusion on when to use or not use the bid item "Excavation for Structures" on overlay projects. In order to remove the confusion the following note is to be added to all overlay projects that only involve removal of the paving block or less.

   Any excavation required to complete the overlay or paving block at the abutments is to be considered incidental to the bid item "Concrete Masonry Overlay Decks".

   If the overlay project has any excavation other than the need to replace a paving block or prepare the end of the deck for the overlay the "Excavation" bid item should be used and the above note left off the plan.

2. For steel girder bridge deck replacements; show the existing exterior girder size including the lower flange size on the plans to assist the contractor in ordering falsework brackets.

3. On structure deck overlay projects:

   Verify desired transverse or roadway cross slopes with Regions considering the changes in bridge rating and approach pavement grade. Although relatively flat by today's standard of an 0.02'/' cross slope; a cross slope of 0.01'/'/0.015'/' may be the most desirable.

   The designer should evaluate 3 types of repairs. (Preparation Decks Type 1) is concrete removal to the top of the bar steel. (Preparation Decks Type 2) is concrete removal below the bar steel. (Full Depth Deck Repair) is full depth concrete removal and repair. The designer should compute the quantity and cost for each type and a total cost. Compare the total cost to the estimated cost of a deck replacement and proceed accordingly. Show the location of (Full Depth Deck Repair) on the plan sheet.

   Contractors have gotten projects where full depth concrete removal was excessive and no locations designated. It would have been a better decision to replace the deck.

4. When detailing two stage concrete deck construction; consider providing pier cap construction joints to coincide with the longitudinal deck construction joint. Also, transverse deck bar steel lap splicing is preferred over the use of bar couplers. This is applicable where there is extra bridge width and the falsework can be left for both deck pours.

5. Total Estimated Quantities

   For all Bituminous Material Overlays:

Surface Preparation for Sheet Membrane Waterproofing	Area of Deck
Sheet Membrane Waterproofing	Area of Deck
HMA Pavement Type E-xx	Check (E-xx) with Region
Asphaltic Material PGxx-xx	Check (PGx-xx) with Region
If Asked for in Structure Survey Report	
Preparation Decks Type 1 & 2	If Blank, call Region
Concrete Masonry, Deck Patching	Use 1/2 Slab Thickness
Sawing Pavement, Deck Preparation, Curb or Joint Repair	If asked for by Region

Table 40.17-1
Quantities for Asphaltic Overlays

40.18 Retrofit of Steel Bridges

Out of plane bending stresses may occur in steel bridges that introduce early fatigue cracks or worse yet brittle fractures. Most, if not all, problems are caused at connections that are either too flexible or too rigid. It is important to recognize the correct condition as retrofitting for the wrong condition will probably make it worse.

40.18.1 Flexible Connections

A connection is too flexible when minor movement can occur in a connection that is designed to be rigid. Examples are transverse or bearing stiffeners fitted to a tension flange, floorbeams attached to the web only and not both flanges.

The solution for stiffeners and transverse connection plates is to shop weld the stiffener to the web and flanges. This becomes a Category C weld but is no different than the terminated web to stiffener weld. If the condition exists in the field, welding is not recommended. Retrofit is to attach a T-section with four bolts to the stiffener and four bolts to the flange. Similar details should be used in attaching floorbeams to the girder.

40.18.2 Rigid Connections

A connection is too rigid when it is fitted into place and allowed to move but the movement can only occur in a refined area which introduces high stresses in the affected area. Examples are welded gusset connection plates for lower lateral bracing that are fitted around transverse or bearing stiffeners.

Other partial constraint details are:

1. Intersecting welds
2. Gap size-allowing local yielding
3. Weld size
4. Partial penetration welds versus fillet welds
5. Touching and intersecting welds

The solution is to create spaces large enough (approximately ¼” or more) for more material to flex thus reducing the concentration of stress. For gusset connection plates provide a larger gap than ¼” and no intersecting welds. For existing conditions it may be necessary to drill holes at high stress concentrations. For new conditions it would be better to design a rigid connection and attach to the flange rather than the web. For certain situations a fillet weld should be used over a partial penetration weld to allow slight movement.

40.19 References

1. *A Study of Policies for the Protection, Repair, Rehabilitation, and Replacement of Concrete Bridge Decks* by P.D. Cady, Penn. DOT.

2. *Concrete Sealers for Protection of Bridge Structures, NCHRP Report 244*, December, 1981.

3. *Durable Bridge Decks* by D. G. Manning and J. Ryell, Ontario Ministry Transportation and Communications, April, 1976.

4. *Durability of Concrete Bridge Decks, NCHRP Report 57*, May, 1979.

5. *Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads, NCHRP Report 243*, December, 1981.

6. *Strength of Concrete Bridge Decks* by D. B. Beal, Research Report 89 NY DOT, July, 1981.

7. *Latex Modified Concrete Bridge Deck Overlays - Field Performance Analysis* by A. G. Bisharu, Report No. FHWA/OH/79/004, October, 1979.

8. *Standard Practice for Concrete Highway Bridge Deck Construction by ACI Committee 345, Concrete International*, September, 1981.

9. *The Effect of Moving Traffic on Fresh Concrete During Bridge Deck Widening* by H. L. Furr and F. H. Fouad, Paper Presented 61 Annual TRB Meeting, January, 1982.

10. *Control of Cracking in Concrete Structures by ACI Committee 224*, Concrete International, October, 1980.

11. *Discussion of Control of Cracking in Concrete Structures* by D. G. Manning, Concrete International, May, 1981.

12. *Effects of Traffic-Induced Vibrations on Bridge Deck Repairs, NCHRP Report 76*, December, 1981.

40.20 Reinforcing Steel for Deck Slabs on Girders for Deck Replacements

Effective Span Ft-In	T=Slab Thickness Inches	Transverse Bars & Spacing	Longitudinal Bars & Spacing	Longitudinal* Continuity Bars & Spacing
4-0 4-3	6.5 6.5	#5 @ 8" #5 @ 7.5"	#4 @ 8.5" #4 @ 7.5"	#5 @ 7.5" #5 @ 7.5"
4-6 4-9 5-0	6.5 6.5 6.5	#5 @ 7.5" #5 @ 7" #5 @ 6.5"	#4 @ 7.5" #4 @ 7.5" #4 @ 7"	#5 @ 7.5" #5 @ 7.5" #5 @ 7"
5-3 5-6 5-9	6.5 6.5 6.5	#5 @ 6.5" #5 @ 6" #5 @ 6"	#4 @ 7" #4 @ 6.5" #4 @ 6.5"	#5 @ 7" #5 @ 6.5" #5 @ 6.5"
2-3 2-6 2-9	7 7 7	#4 @ 9" #4 @ 8.5" #4 @ 8"	#4 @ 11" #4 @ 11" #4 @ 11"	#5 @ 6.5" #5 @ 6.5" #5 @ 6.5"
3-0 3-3 3-6	7 7 7	#4 @ 7.5" #4 @ 7" #4 @ 6.5"	#4 @ 11" #4 @ 11" #4 @ 11"	#5 @ 6.5" #5 @ 6.5" #5 @ 6.5"
3-9 4-0 4-3	7 7 7	#4 @ 6.5" #4 @ 6" #5 @ 9"	#4 @ 10.5" #4 @ 10" #4 @ 9.5"	#5 @ 6.5" #5 @ 6.5" #5 @ 7"
4-6 4-9 5-0	7 7 7	#5 @ 8.5" #5 @ 8" #5 @ 8"	#4 @ 9" #4 @ 8.5" #4 @ 8.5"	#5 @ 7" #5 @ 7" #5 @ 7"
4-3 5-6 5-9	7 7 7	#5 @ 7.5" #5 @ 7' #5 @ 7"	#4 @ 8" #4 @ 7" #4 @ 7"	#5 @ 7" #5 @ 7" #5 @ 7"
6-0 6-3 6-6	7 7 7	#5 @ 6.5" #5 @ 6.5" #5 @ 6"	#4 @ 7" #4 @ 7" #4 @ 6.5"	#5 @ 7" #5 @ 7" #5 @ 6.5"
6-9 7-0	7 7	#5 @ 6" #5 @ 6"	#4 @ 6.5" #4 @ 6"	#5 @ 6.5" #5 @ 6"
4-0 4-3	7.5 7.5	#4 @ 7" #4 @ 6.5"	#4 @ 10.5" #4 @ 10.5"	#5 @ 6" #5 @ 6"
4-6 4-9 5-0	7.5 7.5 7.5	#4 @ 6.5" #4 @ 6" #5 @ 9"	#4 @ 10" #4 @ 10" #4 @ 9.5"	#5 @ 6" #5 @ 6" #5 @ 6"

5-3	7.5	#5 @ 8.5"	#4 @ 9"	#5 @ 6.5"
5-6	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
5-9	7.5	#5 @ 8"	#4 @ 8.5"	#5 @ 6.5"
6-0	7.5	#5 @ 7.5"	#4 @ 8"	#5 @ 6.5"
6-3	7.5	#5 @ 7.5"	#4 @ 7.5"	#5 @ 6.5"
6-6	7.5	#5 @ 7"	#4 @ 7.5"	#5 @ 6.5"
6-9	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-0	7.5	#5 @ 7"	#4 @ 6.5"	#5 @ 6.5"
7-3	7.5	#5 @ 6.5"	#4 @ 6.5"	#5 @ 6.5"
7-6	7.5	#5 @ 6.5"	#5 @ 10"	#5 @ 6.5"
7-9	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-0	7.5	#5 @ 6"	#5 @ 10"	#5 @ 6.5"
8-3	7.5	#5 @ 6"	#5 @ 9.5"	#5 @ 6.5"

Table 40.20-1

Reinforcing Steel for Deck Slabs on Girders for Deck Replacements-HS20 Loading

Max. Allowable Design Stresses: f_c' = 4000 psi, f_y = 60 ksi, Top Steel 2 1/2" Clear, Bottom Steel 1-1/2" Clear, 20 lbs/ft. Future Wearing Surface. Transverse and longitudinal bars shown in table are for one layer only. Place identical steel in both top and bottom layer, except in negative moment region. "Use in top layer for slab on steel girders in negative moment region when not designed for negative moment composite action.